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Engineering News-Record

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April 28, 1927

Counterweighted Cables Raise Span of Carquinez Strait Bridge

Steel Sheetpiling to Repair Approaches to Florida Bridge

Earth Dam on the Deerfield River

Also

Four Pages of "Job and Office" Articles

Timber Logging Trestle in Washington, 204½ Ft. High and 893 Ft. Long

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The Flood Moves South

THE situation on the Mississippi continues to be ominous. As is recounted in detail in the news pages of this issue the largest flood in history is passing down the river. Just how great is the flow is not known at the present time, for volume measurements have not been announced, but the gages near the flood's crest are the highest on record. For some weeks the peril of the lower river will continue, since all the water that goes into the Mississippi must eventually go out its lower valley, no matter how many breaks in the levee system bypass the flow for a time. From Memphis south, then, the next few weeks will be a period of increased watching, of piling sand bags and driving timbers, of loading earth and guarding sandboils. And not until the crest has long passed to the gulf will the valley rest secure and begin again to bind up its wounds. Meanwhile, however, it is not necessary to fall a prey to hysteria. There have been floods before on the Mississippi, and their toll has been greater than this one so far has taken. The dangers so far have been the dangers of submergence, which are bad enough but which are not to be compared with the rush of waters from behind a broken dam. The towns and farms are not meeting a torrent, they are being subjected to a creeping flood that crawls up the sides of the buildings an inch or two, or at most, a foot an hour. Submergence is bad, the losses are great, but until a levee breaks in front of a city the worst horrors of a flood are not experienced. The significance of the present flood has so far lain in its extended area rather than its difference from the Mississippi floods of the past, and every one will do well to read the accounts of its progress with that thought in mind.

Away from the City

IN the old days the American Society of Civil Engineers always held its summer meeting at some resort, far from the busy hum and distractions of the city. Everybody was housed under one roof, the drives and walks and, in more recent years, the golf courses were all convenient and the porches were the meeting places where one could talk to everyone else. It was all very pleasant and some thought that it was useful, that to meet and talk in the pleasant environs of the woods or the sea at least once a year was as worth while as to sit three days in a stuffy meeting room striving against the competition of street cars and motor horns to hear what some earnest fellow engineer was trying to say in a formal paper. But the spirit of reform ruled out these "summer meetings" some years ago and up until this year the quarterly gatherings have been set in large cities. The Asheville meeting last week was therefore an innovation—but it was a successful one. There was enough serious technical discussion to satisfy the conscientious but there was at the same time the atmosphere of vacation and the gathering together for the whole meeting of the whole attendance at one place. Once a year, either in spring or fall or summer, the society can well afford to seek out some such place as Asheville for its meeting. There are plenty of such resorts in all sections of the country so the geographic demands can be met. And the flavor of the meeting so held is so much different from the regular city meetings that the society will benefit.

Meters and Votes

NO ONE outside of Chicago, least of all President Coolidge, will take very seriously Mayor Thompson's crusade to prevent the installation of water meters in that city. True to a pre-election promise, the new mayor lost no time after his inauguration in ordering the water department to stop installation of meters and in having a repeal ordinance introduced in the city council to nullify the metering ordinance passed a year or so ago to comply with one of the War Department conditions to its permit for a continuation of the diversion of Lake Michigan to the Drainage Canal of the Chicago Sanitary District. The session at which the repeal ordinance was introduced was held behind closed doors, but in a statement given out the mayor is reported to have said: "Nobody can make me believe that any President of the United States, or any candidate for President who knows the sentiment of the people of Chicago and who expects to get any votes here, will ever increase the cost of water in our homes 1,000 per cent for the purpose of reducing such use of water so that more can run down the river." When such a threat is made facts are of small consequence. But it will be interesting to watch Chicago once more in her well known rôle of defying the government.

Fruitful Column Study

A DISTINCTLY better understanding of column strength and good design is foreshadowed by the experimental studies which have been in progress for some time at Wisconsin and whose first result are reported in this issue. The work evidently was planned with much ingenuity to clear up a number of matters concerning which there has been uncertainty in estimating column strength. Prior work has given a highly developed knowledge of the action of solid-section columns under direct central load, but their behavior under eccentric loads remained without experimental verification. On this point the new work shows that the resistance of a column in the direction of the two main axes of its cross-section is not the same, the strength in one direction being considerably larger than the usual method of calculation would indicate. This suggests the propriety of applying different working stresses to eccentric loading in the two directions. It is distinctly reassuring on the whole because it

demonstrates the existence of an unexpected safety margin. The other subject of study in the Wisconsin experiments, the latticed column, has been much less fully known than the solid column; nevertheless practice has applied to both the same methods of proportioning sections and estimating strength. It is now shown that in the plane of the latticing the column is several times as flexible as in the other direction, a fact which affects various elements of column calculation. The conventional practice of assuming equal effective moduli of elasticity of the column in both directions and giving it corresponding dimensions in the two directions, is therefore, in need of correction. With respect to the lattice members themselves, however, the tests indicate a satisfactory agreement between observed and calculated behavior, somewhat in disagreement with the few earlier observations recorded. These points of study form the beginning, it is to be hoped, of experimental scrutiny of the other unsettled matters of column action. The results promise to put the subject of steel columns on a firmer foundation.

An Interesting Feat

IN LIFTING two heavy bridge spans a height of some 150 ft. to their place in a cantilever bridge, the builders of the Carquinez Strait bridge performed a feat that will awaken the apreciation of every one who reads the description of the operation. The working speed and reduction of risk obtained by the ingenious use of counterweights, however, are only new pieces of evidence of the high degree of development which steel erection has attained in recent years. The rapid construction of the bridge, once the foundations were completed, is equally significant as one result of private bridge construction. Where each day of earlier completion means an earlier beginning of income to the owners, there is a lively understanding of the serious cost of delay. The fact is that in the case of a free bridge the loss is quite as great, but it is concealed, and the responsible officials are apt to minimize a few weeks' or a few months' delay in opening by saying that it means "only a little inconvenience to a small number of travelers." The fact is, of course, that the loss by delay, or the financial gain from earlier opening, is the same whether the facility is public or private, free or toll. If private toll-bridge enterprise has done nothing else for us, it can be credited at least with the good service of emphasizing the value of rapid completion of construction work so that it may begin earning returns as early as possible.

Co-operative Road Building

THE improvement district as it has existed particularly in the Southern and Southwestern states has not commonly been a notably efficient organization for road building. The record of Arkansas is a spectacular example of what can occur when these legislature-created or court-empowered extra-municipalities supersede the regular local government in highway administration. It is of interest, therefore, to recognize an example of unusual success in the co-operation of some half-dozen Georgia counties through an improvement district in building the so-called coastal highway described in this issue. As a state project the road would have had to await its turn in a state-wide program with limited funds and numerous localities making louder calls for transportation than did these coastal counties. If the development had been left to the individual counties traversed by the road, construction would have been virtually impossible because some of the counties were too poor to pay for their share of the mileage. It was only by such a combination as is described in which the richer counties could help the poorer counties that the highway could be financed and constructed as a continuous operation. But the sum of success required another integer and this was supplied by an excellent district executive. A capable and diplomatic business manager made the improvement district function, kept down county and town jealousies and spent the joint fund efficiently. In brief the Georgia coastal counties improvement district was a success because its business was capably managed. It is history that in road building this has not been too often the case.

Rating the Power of Rivers

THERE has been in the past, and still is to a considerable extent, a great lack of uniformity in the methods of stating the power possibilities of our rivers. Potential power, generally, is stated in horsepower with no qualifications as to whether it is continuous or part-time power, or whether net or gross head has been used in making the computation. Recently, the Geological Survey in its river studies has improved conditions somewhat by giving figures for 90 per cent of the time and 50 per cent. Still more recently the American Engineering Standards Committee undertook to establish a uniform method of rating the power possibilities of streams. A tentative set of rules, drafted by the committee, is given elsewhere in this issue. It deserves careful consideration. The committee is not attempting to produce a substitute for engineering study. What it presents is a method of rating streams which can be applied with facility and which will produce uniform results whose meaning is fully understood by any engineer in this country, and, it is hoped eventually, by any engineer in the world. As a unit of measure, the committee has adopted a compromise between that proposed by those who favor the retention of the term horsepower and those who favor its elimination in favor of kilowatt, giving preference to the latter as the universal term which, because more and more power is being produced and marketed as electricity, has many advantages not possessed by the term horsepower. The power output of undeveloped sites is to be based on 100 per cent efficiency and gross head. This latter procedure is desirable because it does not require stating the methods used in figuring net head or the assumed efficiency of turbines and electric generators.

Modified Hydraulic Fill

BUILDING a second large earth dam within a few years, the New England Power Co. has formalized its modified-hydraulic-fill method to the notable degree described and diagrammed in this issue. Compared with the full water sorting and carriage of materials of the true hydraulic fill process, there has always existed, despite many successful operations, a lingering doubt, in using modified hydraulic fill, of the homogeneity of the core fill. With hydraulic giants directed from the edges of the fill it seemed too much a personal equation of the nozzle operation whether the core was or was not shot

through with intrusions of coarse material from the berms. In the dam work described in this issue and in the previous Davis Bridge dam for the same stream development, the hydraulicking was done largely from a float in the core pool. This insures that the material coming to the core is brought there by true flotation and not driven there by the force of a hose stream. With reasonable care by the monitor men, then, only the finer material ought to get into the core, and a water placed dam embankment of the first order be secured.

Traffic Administration

IT IS refreshing to learn of the conviction recently of the village constable of a community near Chicago, who for some years maintained a local road despotism and in its course committed a murderous assault on a party of motorists whom he tried to terrorize and tax. At the same time the case is a stimulating reminder of the importance of traffic administration. Road construction and the planning of the many auxiliary structures and arrangements necessary for efficient traffic movement are the first two of the three elements of traffic provision; but the third, traffic administration, may prove to be quite as important as the other two. Certain it is that if local officials set up conditions which discourage or penalize traffic, the efficiency of the road system centering in that community is decreased and the public's investment in the system is impaired. In so far as highway construction aims to assure efficient movement of traffic, then, traffic administration is of direct importance to highway engineers.

Earthquakes and Building Codes

REVISION of the San Francisco building ordinance is being urged in order to lessen danger of earthquake damage by making construction more rigid. Soon after the 1906 quake San Francisco adopted a building code that said nothing about earthquakes but specified for buildings of certain heights a wind-pressure allowance of 30 lb. per square foot. This doubtless gave assurance against serious damage to structures from even fairly heavy earth tremors, and did so without mention of earthquakes—just then regarded as particularly undesirable. But 30-lb. wind loads rarely occur. In time, the lessons of the earthquake were more or less forgotten and there was clamor about the "unreasonable wind-load requirements." The ordinance was several times revised and now provides for a wind load of only 15 lb. per square foot.

Perhaps the 30-lb. wind load was too high to permit of consistent enforcement. While it was in effect its requirements were frequently evaded. However, it was unwise to cut the wind load down to 15 lb. with no compensating provision for stiffness or rigidity to lessen earthquake damage. This reduction was a victory for those whose dominating motives were cost saving and competition with other cities for business. But the change has brought a reaction and some builders have been voluntarily using more rigid specifications. In view of the present more sensible attitude toward earthquake risks the psychological time has come to put into the building code in San Francisco and elsewhere provisions that frankly recognize the possibilities of earthquake stresses and that make allowance for them.

The unfortunate part of the situation is that engineers are not in entire accord as to just what is the best kind of protection against earthquakes. Some favor maximum economical rigidity throughout the entire structure. Of this group some would design the structural frame to furnish the entire resistance to shock while others would make use of the exterior walls and special interior bracing walls to assist the frame. Since 1906 some have held the belief that buildings should be braced only by rectangular paneled bracing, i.e., bracing without diagonals or with diagonals only in alternate panels. Typical of this is the Sather tower on the Campus at Berkeley, which is designed to resist earthquakes (*Engineering Record*, March 14, 1914, p. 312). Quite recently a few engineers have proposed a high ceiling in the first story without lateral bracing of the long, first-story columns, the theory being that, where such construction is possible, the earthquake damage will be lessened and will be entirely confined to the first story. One well known engineer goes so far as to say that a structure designed to resist a 30-lb. wind load would have withstood the most violent earthquake that has occurred in the United States in the past fifty years. On the whole, this confusion in engineering opinion, while not creditable to the profession, is not surprising when it is realized that only recently have engineers begun to study the problem in earnest.

Admittedly it is not practicable to design to withstand the most violent earthquakes even if the stresses were known, but surely there is some reasonably high degree of protection that can be secured for a moderate increase in cost. Pending the outcome of study and research now being devoted to this subject, any approach to the ideal would be better than the present practice of designing almost, if not entirely, without regard to earthquake shocks. The hysteria over earthquakes and the belief that they occur only in certain localities has been largely dissipated in recent years. It is now known that an average of 30,000 earth tremors are recorded annually on seismographs in different parts of the world, and at least once a year there occurs somewhere a major quake, that is, one of intensity above 7 or 8 on the Rossi-Forel scale. Hence there is now no occasion in San Francisco or elsewhere to veil under the guise of needed wind-load allowance the requirements for substantial construction that will afford the maximum earthquake protection for reasonable cost.

Our practical knowledge of how buildings behave when subjected to earthquakes of major intensity is limited to the case of buildings of relatively low height; the tallest buildings in San Francisco in 1906 were approximately 18 stories high and the highest structures in Japan are not over 100 ft. The effect of a destructive earthquake on a modern office building 30 to 50 stories high is entirely in the field of theory, if not of conjecture. Thus far, however, wherever modern building construction has been subjected to earthquake, well built and rigid structures have suffered but little damage, and the danger of life in such structures has been slight as compared to the complete destruction with appalling loss of life that has occurred in flimsy structures. It is a safe conclusion that, while we need much more research on the stresses to which earth tremors subject a modern structure, yet the real danger to the great majority of buildings lies not so much in earthquakes as in flimsy construction. Revision of building ordinances in this spirit will increase the earthquake resistance of city buildings.

Another Earth Dam Built on Deerfield River

A 100-Ft. Dam Built by Modified Hyrdaulic-Fill Method in One Working Season—River Bypass and Intake Both Carried Through Dam in Concrete Conduits

By Lester Gurney
Resident Engineer, Sherman Power Construction Co.,
Shelburne Falls, Mass.

ANOTHER step in the development of the Deerfield River for power has been completed by the New England Power Co. in its Sherman Development located about a mile upstream from the village of Monroe Bridge in Franklin County, Massachusetts. Work was started on the project in September, 1925, and the power station was put into service in January, 1927.

The Sherman Development is on the upper reaches of the river. It has a drainage area of 234 square miles. Above it are the Davis Bridge dam, a high earth-fill dam, described in *Engineering News-Record* Jan. 24, 1924, p. 142 and Feb. 7, 1924, p. 235, with a drainage area of 154 square miles, and Somerset dam with a drainage area of 30 square miles. The Somerset reservoir has a capacity of 62,000 acre-ft. and, since it was put in operation in 1914, has completely regulated the flow at that point. The Davis Bridge reservoir has a capacity of 116,000 acre-ft. It was filled in May, 1924, and it is estimated that water will spill from it on an average of about once in three years.

The Sherman Development consists of an earth dam about 100 ft. high and 700 ft. long, constructed by the modified hydraulic fill method. The water intake, penstock and power house are located on the left side of the river as shown in Fig. 1. The spillway and the bypass conduit are on the right bank.

Preliminary Work—Within the reservoir site were two miles of railroad, two miles of highway and a cemetery from which 110 bodies had to be moved. These all had to be relocated before the dam could be completed. Also, a bypass had to be built to carry the river through the dam during construction.

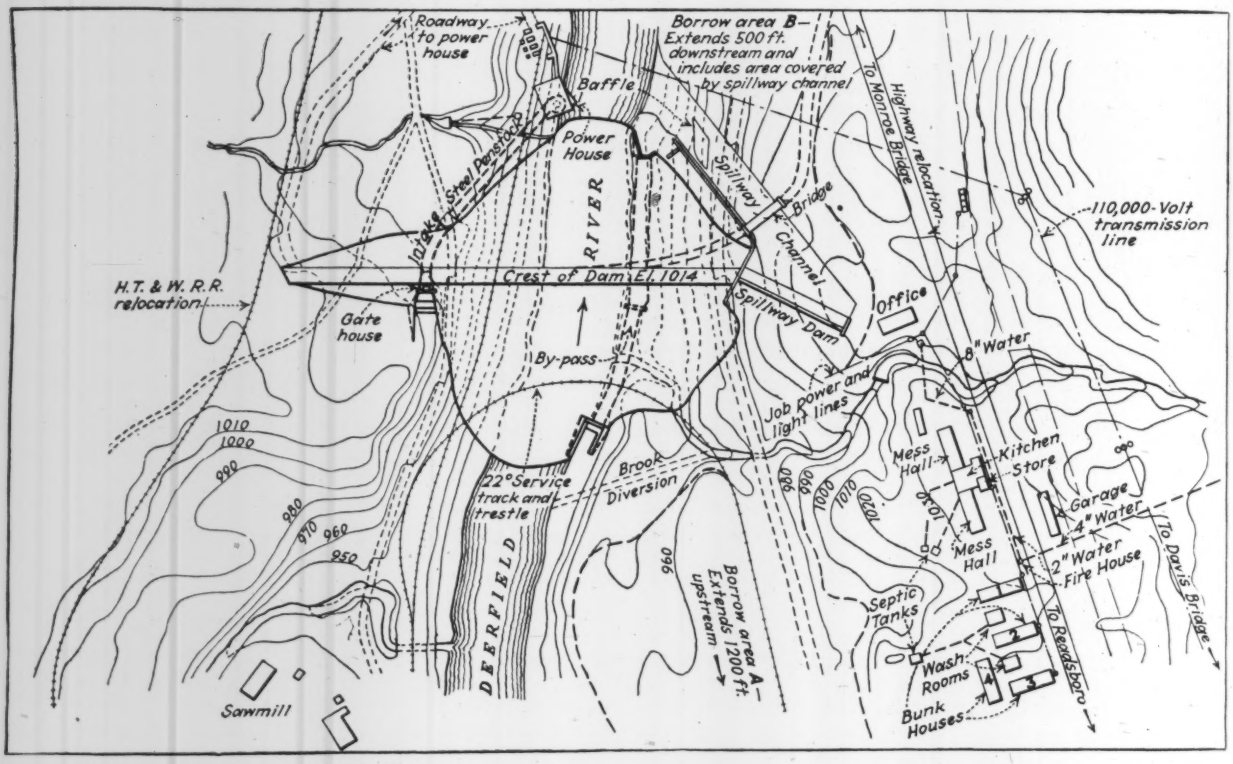


FIG. 1—SHERMAN POWER DEVELOPMENT SHOWING LOCATION OF STRUCTURES

The first step was the construction of a 300-man camp on the west side of the river immediately upstream from the dam and adjacent to the highway relocation. As soon as it was ready for use, work was begun upon the bypass conduit. The location of the conduit is shown in Fig. 1 and the details in Fig. 3. Its upstream portion is located on a hardpan, composed of a rock flour resembling clay with scattered boulders, while the downstream portion is founded upon a ledge. The design of the conduit was modified to suit the different types of foundation. Excavation for the bypass was started Oct. 1, 1925, and it was completed by Jan. 1, 1926, despite the difficulties of winter work and the placing of 4,000 cu.yd. of reinforced concrete in cold weather.

Railroad Relocation—While the construction of the

bypass was in progress, work was started on the relocation of two miles of standard-gage railroad. This involved some 50,000 cu.yd. of earth excavation, 10,000 cu.yd. of rock, and 7,000 cu.yd. of fill. Traffic was diverted to the new line early in May, 1926.

Construction of the Dam—In order to proceed with the placing of materials in the dam as early as possible in the spring, it was necessary to strip the site during the winter months. To do this, a cofferdam was built in connection with the curved service track trestle shown in Fig. 1 as being on the upstream side of the dam site. This cofferdam diverted the river flow to the bypass conduit and made it possible to unwater the dam site. It was provided with three waterways to take care of spring flood flows in excess of the bypass capacity. the trench was excavated in ledge as far as the spillway retaining wall.

Excavated materials were loaded into 4-yd. cars on tracks carried up with the toe fill and hauled by cable.

A total of 10,000 cu.yd. of material was stripped from the site. Fig. 4 shows the typical cross-section of the embankment at the conclusion of this stage of the construction, including the fill at the downstream side to form the central pool.

The shovel was ramped out of the river bottom over the toe fill before the spring flood arrived. The excess flood water was allowed to spill through the waterways in the cofferdam filling the stripped middle third area and then running out through a low area left in the rock fill adjacent to the downstream end of the bypass.

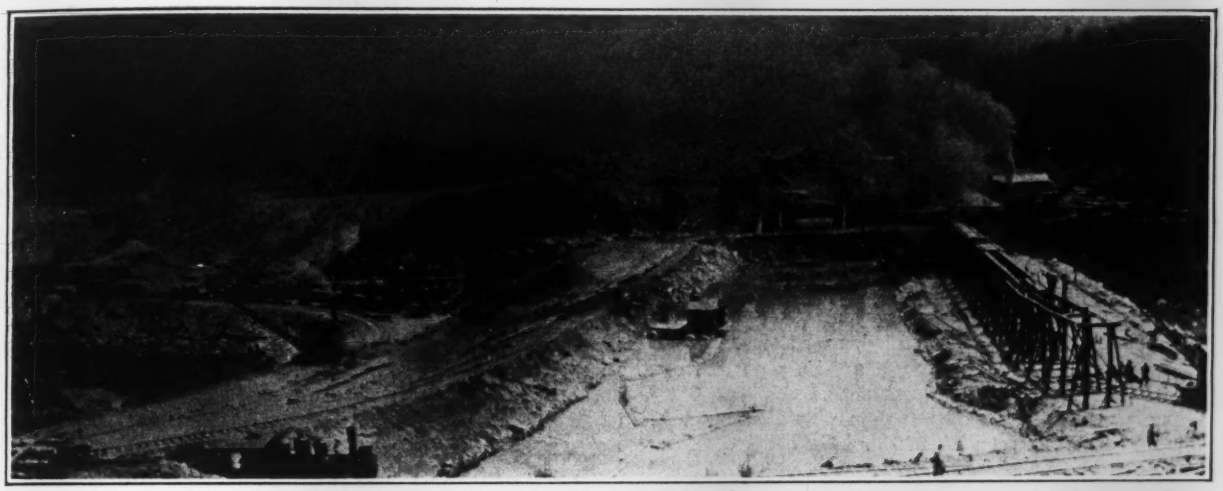


FIG. 2—STRIPPING THE DAM SITE PREPARATORY TO FILLING
Service trestle and cofferdam at right, shovel and dragline stripping in foreground, and toe fill at left. Bypass conduit at far side of pool—May 20, 1926.

As soon as the cofferdam was closed a steam shovel was placed in the river bed and the materials stripped from the outer thirds were spoiled downstream of the site while boulders and materials from the middle third were deposited to form the downstream toe of the dam. The stripping in the area covered by the outer thirds of the embankment was confined to the removal of vegetation, stumps and loose rock. In the middle third, however, the stripping was carried to impervious material and a core trench about 30 ft. wide and 4 ft. deep was excavated along the median line of the embankment and subsequently extended up the east side of the valley by means of a dragline. On the west side

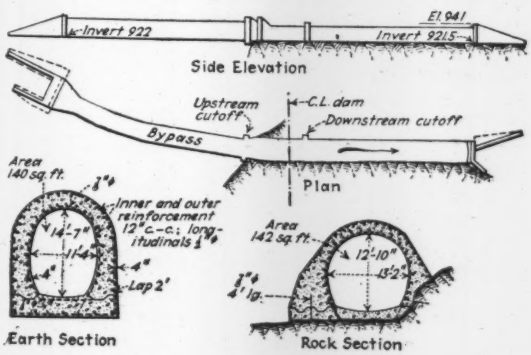


FIG. 3—DETAILS OF THE BYPASS CONDUIT

A very gradual spring break-up delayed the start of placing materials in the dam about a month. The first train load was placed from the 940 level upstream

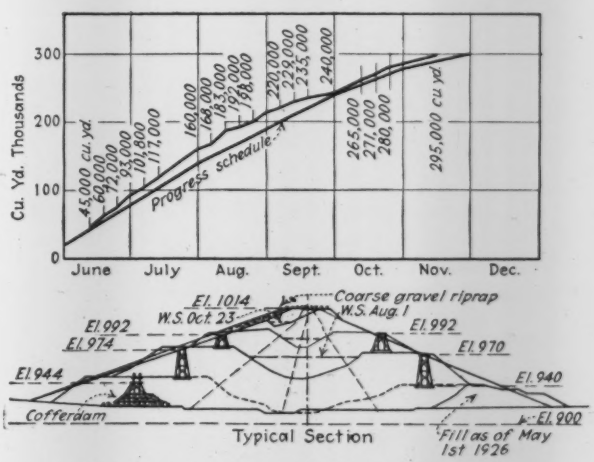


FIG. 4—PROGRESS CHART ON EARTH FILL

trestle, the latter part of May, and this operation continued until Nov. 15 when the 295,000 cu.yd. were in place and the dam completed. Fig. 4 shows the embankment construction progress.

Materials for the embankment were taken from areas

marked *A* and *B* on Fig. 1, except the last 30,000 cu.yd. which were borrowed adjacent to the railroad relocation immediately downstream of the dam.

Mechanical analysis of samples from the test pits in the borrow areas indicated an effective grain size of 0.09 mm. Analysis of the 202 core samples taken during construction is shown in Fig. 8.

Water from the central pool was used in the sluicing operations. A two-stage 750-gal. motor-driven centrifugal pump, mounted on pontoons in the central pool, was equipped with a 7-in. hydraulic giant with 2-in. nozzle. A similar pumping plant was located in the river and four lines of 3-in. fire hose furnished makeup water to maintain the central pool at the proper elevation and for additional sluicing from the top of the dikes. Stable beaches were maintained and no slides occurred. The central pool was carried to El. 1,000 after which sluicing was discontinued, the pool filled, and the excess water drained into the river through a 6-in. pipe laid on the upstream face of the embankment.

The upstream face of the dam was riprapped for a depth of 3 ft. between El. 990 and the crest with coarse bank gravel.

Earth moving equipment consisted of two railway type steam shovels, one steam shovel used as a dragline, four construction locomotives, one 25-ton locomotive crane, sixteen 12-yd. dump cars and about two miles of standard-gage track.

The highway relocation, which follows the west side of the reservoir, comprised some 36,500 cu.yd. of earth and 1,000 cu.yd. of rock excavation; 18,500 cu.yd. of borrow for fill and a 23-ft.-span concrete highway bridge. The main highway is connected to the power house by a private roadway crossing the spillway channel on a 70-ft.-span steel girder bridge. The roadway follows along the crest of the dam to the railroad relocation and thence down to the east side of the valley to the power house.

Spillway—The west end of the dam is retained by a reinforced-concrete wall 20 ft. high founded on ledge extending from a point 40 ft. upstream of the center line of the dam to the downstream toe. A concrete spillway dam extends west from this wall a distance of 180 ft. The spillway crest is at El. 998. A spillway channel excavated in ledge will convey flood water into the river at a point immediately downstream from the dam and opposite the power house. This channel has a minimum width of 50 ft. and the upstream portion a gradient of 6 per cent which is increased to 12 per cent in the downstream portion. A scale model of the spillway channel and power house site indicated the necessity of placing a concrete baffle pier at an angle of about 45 deg. with the spillway channel and located about 75 ft. from the discharge end. This baffle will spread the discharge and aid in deflecting a portion of the flow into a direction parallel to the river channel.

A total of 33,000 cu.yd. of earth and 16,000 cu.yd. of rock were excavated in connection with the spillway; 3,000 cu.yd. of concrete were placed in the spillway dam and retaining wall, and 420 in the baffle pier.

FIG. 5—PROGRESS AFTER ONE MONTH OF FILLING
Retaining wall in foreground, monitor raft beyond and two make-up lines adding to central pool. Power house under construction at right—June 18, 1926.

Intake—The intake conduit is of the same cross-section as that of the downstream portion of the bypass conduit and conveys the water through the dam to the

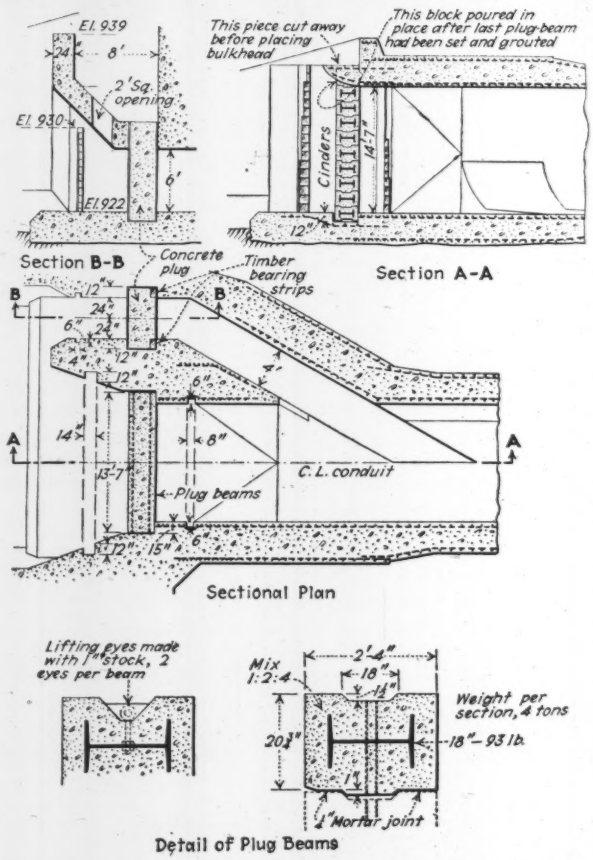


FIG. 6—ARRANGEMENT OF BYPASS FOR RAPID CLOSING

penstock. The intake structure and rack supports required some 8,000 cu.yd. of earth and boulder excavation and 1,750 cu.yd. of reinforced concrete.

This portion of the work was of necessity carried on simultaneously with the construction of the dam and was completed without delay to the embankment progress.

A 14-ft. diameter Dow disk-arm motor-driven valve located at the upstream end of the intake conduit is operated from the switchboard of the power house. The valve disk is built of structural steel, filled with concrete.

Immediately downstream of the face of the dam the concrete conduit changes to a 13-ft. diam. steel penstock 235 ft. long. The penstock was shipped in 8-ft. lengths, three sheets to a section, and field assembled and riveted into 24-ft. sections. It is supported on concrete cradles at 24-ft. centers and was backfilled after receiving two field coats of coal tar paint.

Bypass Closure—Upon the completion of the dam and appurtenant structures the intake end of the bypass was closed and the reservoir allowed to fill. The bypass was provided with two openings as shown in Fig. 6. For the closure the Davis Bridge plant was shut down and the stream flow reduced to about 50 sec.-ft. Stop planks were placed in the grooves up and downstream of the large opening and the water pumped out and the groove for the concrete stoplogs carefully cleaned. Cement mortar was spread on the invert of the groove and the first stoplog lowered in place. The precast stoplogs were 20x28 in. and 13 ft. 7 in. long. They were reinforced with 18-in. I-beams for their entire length. The top face of each of the stoplogs was provided with a 1½-in. groove, 10 in. wide, into which a tongue on the bottom face of the adjacent log fitted. Each log was buttered with cement mortar about ¾ in. thick and screeded to receive the tongue. The top log was joined to the main structure by 2½ ft. of concrete which completed the closure. The area between the concrete stoplogs and the upstream stop plank was filled with cinders. Stoplogs and materials were handled with a steam shovel equipped as a dragline. Work was started at 7:00 a.m. and completed at 5:30 p.m. The following day stop planks were placed in the groove upstream of the small opening to permit cleaning the recess for the plug. This opening was closed by lowering a precast concrete plug with a timber bearing edge into the groove provided and filling the area between the plug and the stop plank with cinders. Both closures were made under practically dry conditions and subsequent inspections on the downstream side have failed to indicate any leakage. Subsequently, the Davis Bridge plant was started and the reservoir allowed to fill. Twenty-two hours after closing the small opening the plant was put in operation.

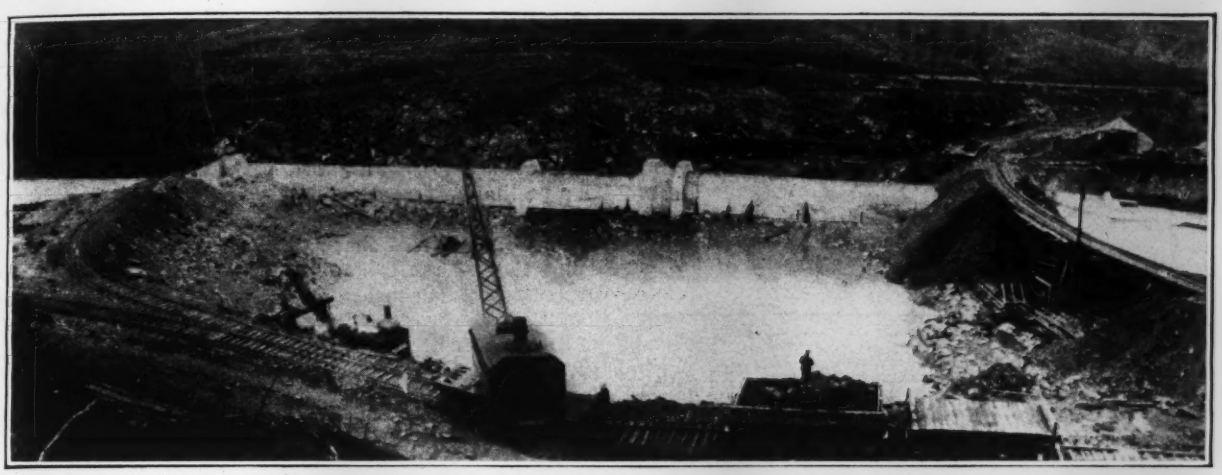


FIG. 7—TRESTLE BEING BUILT FOR ADDITIONAL FILL
Monitor on raft at left bank of central pool. Dragline trimming up-stream slope of fill—Aug. 1, 1926

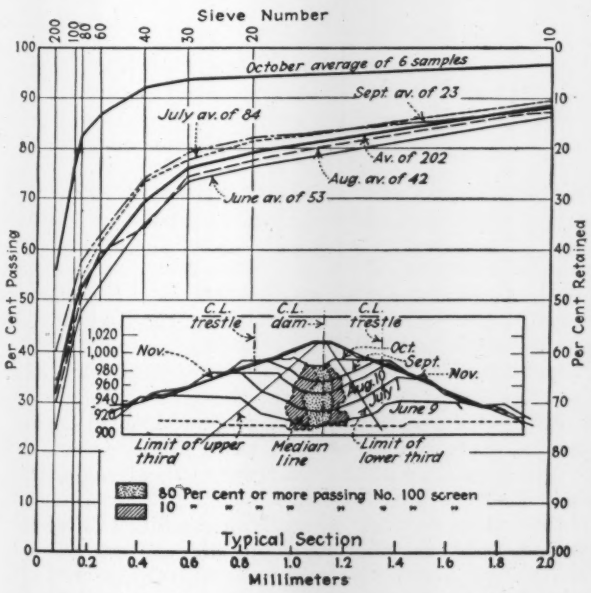


FIG. 8—MECHANICAL ANALYSIS OF CORE MATERIALS

Power House—Excavations for the power house and tail race were started May 15 and totaled 4,500 cu.yd. The building is 58x46 ft. in outside dimensions. Its concrete substructure was completed Aug. 15. The superstructure is of steel-brick construction with concrete trim and tile roof.

Hydraulic equipment consists of a 9,500-hp. vertical-runner turbine with steel plate scroll case, operating at a normal head of 78 ft. at 180 r.p.m., directly connected to a 9.000-kva. generator. The turbine was built

by S. Morgan Smith and the generator by Westinghouse.

Electrical Equipment—The station is automatic in operation and is controlled from the Davis Bridge power house. Energy is generated at 2,300 volts, stepped up to 110,000 volts and transmitted to the system through the Davis Bridge station. The line transformers and disconnecting switches are located immediately downstream from the power house.

The line transformers are mounted on trucks and may be brought into the power house on a transfer

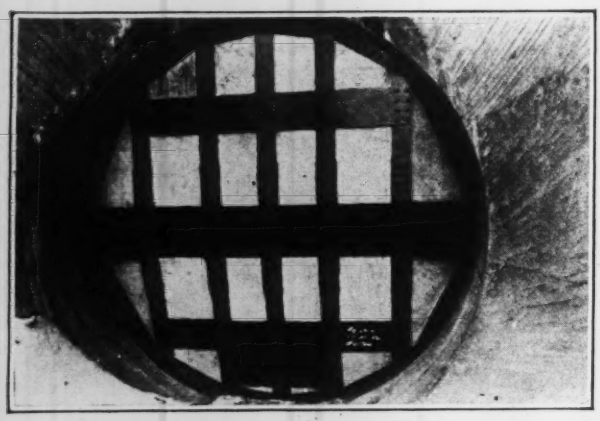


FIG. 9—14-FT. DIAM. VALVE AT UPPER END OF INTAKE CONDUIT
Disk-arm valve built of structural steel and filled with concrete.

track through a rolling door in the downstream end of the building. A 30-ton crane is part of the station equipment and can be used in placing transformers into power house basement for cleaning and repairs.

All work in connection with the development was done by the Sherman Power Construction Co., a subsidiary of the New England Power Co. G. S. Hewins is president of the construction company, J. V. Downing was superintendent of construction, and the author resident engineer.

Checking Tests of Hardness of Metals

The U. S. Bureau of Standards recently issued Technological Paper 334 in which is given the relationship between the results obtained by two widely used methods of measuring the hardness of metals, the Brinell method and the Rockwell method. Tests were conducted on a great number of ferrous and non-ferrous metals. Convenient semi-experimental formulas have been derived by means of which the Rockwell or the Brinell hardness number may be computed from the other number with an error of less than 10 per cent. Similar formulas were obtained also for tensile strength, which can be computed from the Rockwell number to an accuracy within 15 per cent. The value of these relationship formulas will lie in the fact that for the testing of tensile strength, which tests are often expensive because they require testing machines and often a sufficient number of specimens to represent all the parts of a structure, there may be substituted the simpler indentation test of hardness by one of the methods named. Comparison of the hardness or the tensile strengths of two materials for one of which the Rockwell and for the other of which the Brinell number is known is made possible by the formulas in paper 334.

Column Strength Under Eccentric and Shear Loading

Tests at Wisconsin Throw Light on Influence of Shear, Eccentricity, and Strength of Lattice and Battens

NEW RESEARCH in the field of columns was initiated three years ago under the auspices of the American Society of Civil Engineers by a committee headed by Prof. F. E. Turneaure, of the University of Wisconsin. The initial series of tests made by this committee has recently been completed, sufficiently for preliminary publication of the results. The plan of these tests was based on a survey of column literature which led to the conclusion that direct attempts to deduce a column formula have been carried as far as is of present value, and that other features of columns than length ratio have at least as great an influence upon practical column strength The tests accordingly were devised to investigate certain of these influences in a preliminary way.

The primary factors considered in the new test series are the influence of shape of section, method of cross-connection of the column elements (solid web, lattice or battens), influence of eccentricity, and relation of longitudinal shear to the strength of the column and of its material. A brief account of the findings was given in the report of the annual meeting of the American Society of Civil Engineers, *Engineering News-Record*, Jan. 27, 1927, p. 156, but it was not possible at that time to exhibit any of the results. Material since made available by Professor Turneaure makes it possible to present the accompanying groupings of results,

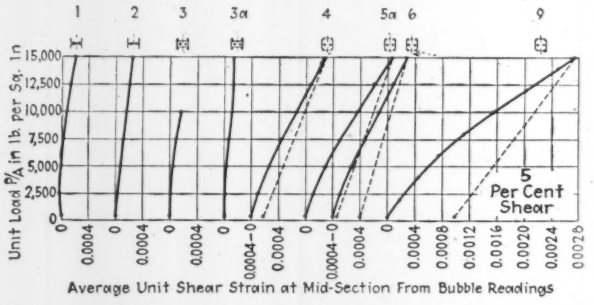


FIG. 1—SHEAR DISTORTION OF VARIOUS COLUMN TYPES PLOTTED AGAINST LOAD

Figs. 1 to 3, together with a brief explanatory statement supplementing that given in the report quoted.

As a preliminary investigation of the method of lateral connection of column components, columns of I-section and two-channel latticed section were tested as beams, under lateral bending load in the direction of web or lattice. It was found that the latticed columns deflect considerably more than solid-web columns, and therefore have greater lateral flexibility. It is concluded that any calculations of column strength involving the modulus of elasticity, as by the Euler or the Gordon formula, should use a lower value of modulus of elasticity for strength against buckling in the direction of the lattice than for strength in the direction of the channel webs or other solid elements of the column. This has not been done in the past, with the result probably of leading to insufficiently rigid proportioning of

latticed columns in the direction of their lattice members.

Initial Eccentricity—Careful measurements of the initial eccentricity of a number of columns were made as follows: (1) eccentricity from rolling was determined from measurement of end cross-sections; (2) crookedness was determined by direct measurement; (3) eccentricity at mid-section was found by trial loading, strain-gage measurements on all four edges, and re-centering; (4) obliquity of ends was determined from the shimming required.

COLUMNS UNDER OBLIQUE LOADS
(5 Per Cent Shear)

COL. NO.	TYPE	MAKEUP	AREA IN.²	I IN.⁴	LENGTH	$\frac{L}{r}$	MAX. P/A, LB/IN.² 5% SHEAR	COMPUTED UNIT * STRESS LB/IN.²	TENSILE Y. P. OF STEEL LB./IN.²
1		5" x 3½" x ⅝" Ls 10" x ⅝" Pl.	13.19	233	77	19	28,000	44,900	Ls 38,400 Pl. 40,900
2		10" 49.5-lb. H-col.	14.15	256	89.5	21	26,700	42,800	36,200
3		12" x ¼" Pls. 10", 15 lb. [	14.67	288	89.5	20	25,900	41,500	[35,500 Pl. 35,000
3a		"	14.43	282	89.5	20	24,900	39,800	[36,170 × Pl. 34,710 ×
4		10", 20-lb. [2¼" x ⅜" L.B. ¼" S. pls.	11.52	334	119	22	22,800	36,500	[32,800
5	"	" But 2¼" x ¼" L.B.	11.3	334	119	22	22,000	35,200	"
5a	"		11.62	340	119	22	22,300	35,700	[34,630 ×
6	"	" But 2½" x ⅜" L.B. and ⅜" S. pls.	11.30	324	119	22	24,000	38,400	[32,800
7		10", 20-lb. [¼" S. pls. 2, 9" x 12" x ¼" batten pls.	11.84	351	119	22	10,000	16,000	[34,630 ×
8	"	" But 4 batten pls.	11.84	349	119	22	15,000	24,000	"
9	"	" But 8 batten pls.	11.88	352	119	22	20,700	33,100	"

* Computed unit stress on backs of web at column ends. × Mill tests

FIG. 2—COLUMNS UNDER OBLIQUE LOAD

Strength under 2 per cent and 5 per cent shear. Column strength of solid, latticed and battened columns compared with yield-point of material.

The rolling eccentricity was in all cases very small, ranging from 0.01 in. to 0.05 in. in ten 8-in. H-columns varying from 39 to 307 in. in length. The amount of this eccentricity seemed to be quite unrelated to column length, a short column being just as likely to have the larger value of eccentricity as a long column.

Crookedness was considerably larger especially in the longer columns, and as a rule was greater normal to the web than in the direction parallel to the web. For the H-columns tested the maximum eccentricity was about $L/750\ r$.

Subsequent tests of column strength under intentional eccentricity of loading, in which lateral deflection was measured show: that the behavior of columns whether under intentional or initial eccentricity, follows very precisely the so-called secant law, the well-known formula based on the combined extreme fiber stress as determined by the actual deflection of the column under its compressive load.

Diagonal Load; Shear—Under opposite eccentricities of loading at the two ends of the column, which subjected the column to a diagonal load introducing a large shear, the columns behaved in the manner summarized in the diagram, Fig. 1, and the tabulation, Fig. 2. It will be seen that the four solid-section columns show relatively small lateral distortion and no shearing set; the next three, which are latticed sections, show shearing distortion about three to four times as large, and a distinct shearing set, the latter indicating either shearing overstrain of the main members or slip in the lattice rivets.

The last column, representing a battered section shows an apparent shear strain two to three times as large as that of the latticed columns, and a correspondingly increased shearing set. In the latter case it is to be remembered that bending of the main channels between the batten plates contributes to the lateral distortion, which therefore is not to be considered as a pure shearing distortion. However, it is plain that both latticed and battened columns are more flexible under lateral shear than solid columns, and therefore, as already indicated, a lower value of modulus of elasticity should be used in column formulas for such columns.

Column strength under this diagonal loading was affected in a quite different way from shearing deflection. The table in Fig. 2 shows, by the maximum unit stress developed under a 5 per cent shear load, when compared with the yield-point value of the material (last column), that in the solid-section and latticed columns the maximum stress developed was well above the yield-point, and the margin of excess was about as much in the latticed columns as in the solid-web columns. The batten-plate columns, however, show much lower load capacity for the two columns with two and four batten plates, respectively, while eight batten plates brought the fiber stress at maximum up to the neighborhood of the yield-point of the material. These results illustrate the inability of batten-plate connections to give the column great shearing strength, unless the battens are set prohibitively close.

8-IN., 32-LB. H-COLUMNS UNDER ECCENTRIC LOADS
($r_1 = 3.40$ in., $r_2 = 1.98$ in.)

COL. NO.	TYPE FOR ALL COLS. TESTED IN THIS LIST	AREA SQ. IN.	LENGTH IN.	$\frac{L}{r_1}$	$\frac{L}{r_2}$	MAX. UNIT LOAD LB./IN.² $\frac{e_1 c_1}{r_1^2} = 1$ *	MAX. UNIT LOAD LB./IN.² $\frac{e_2 c_2}{r_2^2} = 1$ **	DEFLECTION AT MAX. LOAD, IN.	COMPUTED UNIT STRESS MAX. LB./IN.²	TENSILE YIELD POINT LB./IN.²
H1		9.40	39.1		20		26,900	Not found	53,800	37,900
				12		20,700 †			41,400	
H6		9.61	39.6		20		28,800	0.25	64,900	38,030
H2		9.37	99.0	29		19,950		0.41	42,700	37,900
H7		9.63	99.0		50		21,650	1.12	68,100	38,030
H3		9.28	168.2	50		17,950		1.11	42,600	36,800
H8		9.63	168.2		85		16,150	1.60	58,600	41,640
H4		9.44	237.7	70		15,100		1.48	37,800	38,500
H9		9.32	237.5		120		10,640	1.95	42,400	41,970
H5		8.97	306.9	90		12,600		2.55	36,200	36,600
H10		8.92	306.9		155		7,750	2.85	38,100	38,110

* Eccentricity at end of column, $e_1 = 2.93$ in.
** " " " " " $e_2 = 0.98$ in.
† Straightened after previous test ($e_2 = 0.98$ in.) and retested with $e = 2.93$ in.

FIG. 3—ECCENTRIC-LOAD TESTS OF I-COLUMNS

In these tests the action of columns with load eccentric in the direction of the web is compared with the action under load eccentric normal to the web. For the latter loading the column developed a much higher nominal stress in the outer fiber, as shown in the column preceding the last.

Eccentric Loading—Load tests under eccentric loading are summarized in the tabulation Fig. 3. Each type of section was subjected to loading eccentric first in the direction at right angles to the web, and second in the direction of the web. The calculated edge stress at maximum load exceeds the tensile yield-point of the material (last two columns) in nearly every case, the difference being greatest for short lengths and progressively smaller for longer lengths, disappearing in the region of $l/r = 100$.

Equally interesting is the fact that the edge stress at maximum load is considerably larger in the weak

direction of the member than in the strong direction. In other words, columns H-3 and H-8, which were of the same dimensions and length but were tested under eccentricity in the strong and weak direction, respectively, reached the limit of their strength at stresses of 42,600 and 58,600 respectively. This appears to indicate that when the entire face of the flange is subjected to a given stress simultaneously, the maximum unit stress which the section can take is considerably smaller than when only the tips of the flanges are subjected to the extreme stress. In practical application, this fact would permit more liberal eccentricity calculations in the weak direction of a column than in the strong direction.

A full report of these and future tests is to be presented by the committee in its 1928 report, next January. The results and inferences given above are to be considered as preliminary and subject to correction and modification, but they are made public in advance because of their importance to structural analysis and design.

French Test on Alumina-Cement Concrete Shows Study Needed

IN *Le Genie Civil*, March 12, 1927, E. Freyssinet, the noted engineer of the Corps des Ponts et Chaussées, now retired, together with one of the engineers of the same corps, A. Coyne, presents a long paper calling attention to some of the peculiarities of concrete made out of alumina cement. They had observed in a certain structure in which "ciment fondu," a French alumina cement, had been used that there was a great difference in the structure, some parts of it being quite sound and others being made up of patches of a softer material of a different color. The normal and good concrete was apparently of dark gray color, lightly tinted with blue or brown, whereas the poorer concrete, which did not give a sound ring under the hammer, was a greenish blue-black or a reddish yellow, a dull earthy color. In the effort to discover what was the cause of the difference, the engineers made a series of tests with alumina-cement concrete gaged with both fresh water and sea water and also gaged under different temperature conditions.

As a result of these tests, they found that alumina-cement concrete gaged with sea water has a strength up to 67 hr., at least, of about half of the strength of the same concrete gaged with fresh water; second, that the concrete made with alumina cement is apt to be seriously affected in strength when the temperature of the water or of the material in the mixing rises above a minimum, say, equal to the temperature of the materials when strongly heated by the sun's rays, though this divergence in strength may be made up at a later date.

The investigators conclude, therefore, that, while alumina-cement concrete may be perfectly satisfactory, it is desirable to submit it, in addition to the regular laboratory tests, to some special tests to determine the variation of temperature during setting under different conditions of initial temperature, quantity of gaging water, etc. They add that it is essential not to use alumina cement under conditions where the materials or the final mix are heated beyond the normal heat applied to concrete. They feel that an ample quantity of gaging water should be used and that the concrete should be kept very wet while it is setting. Finally they call attention to the fact that the variation in color may be an excellent indication of the structural condition of the concrete, and advise that no sea water be used in the gaging.

Tentative Standards for Rating Power Resources of Rivers Adopted

AS a result of the confusion which arose at the World Power Conference in London in 1924 over the various ratings used in different countries to state the power resources of rivers, the American Engineering Standards Committee, in October, 1924, appointed a sectional committee to formulate rules for stating the potential power resources of rivers and the capacities of water power plants in the United States. N. C. Grover, chief hydraulic engineer, U. S. Geological Survey, Washington, D. C., was appointed chairman of this committee and forty engineers, representing the consulting engineering, financing and management groups, the manufacturers, state and municipal organizations, and the federal government, were appointed as members of the committee. In November, 1925, this committee sent out a questionnaire to get the views of those interested in the subject, and after a number of meetings of groups of engineers on the committee, held in Washington, New York and Chicago, has formulated a tentative set of rules based on the following conclusions:

(a) The duration curve based upon records of daily discharge of at least five years, and preferably longer, should be used in the determination of the rates of flow that are made a part of the rules.

(b) One hundred per cent efficiency at gross head of the site should be used, leaving to the judgment of the user of the data the determination of the proper efficiency factor or factors applicable to any situation.

(c) Energy should be expressed preferably in kilowatts with the horsepower equivalent in parentheses.

The rules have been adopted tentatively with the idea that they be broadly circulated in order to obtain a general discussion before final adoption. The committee points out that they are not intended as bases for design or operation or to take the place of detailed and complicated investigations or studies necessary to determine the feasibility of a site or the best manner of development. The rules are:

Undeveloped Site or Stretch of River—The rating of undeveloped sites will show gross energy based on 100 per cent efficiency and gross head, and will be expressed in kilowatts with the horsepower equivalent in parentheses, except in instances where it is desirable to make comparisons on the horsepower basis when the gross horsepower will be given and the equivalent kilowatts in parentheses. In this rule gross head is the difference in elevation of water at the point of diversion or pool created by the dam and the return of water to the stream; or if a stretch of river be considered on which surveys are not available to determine the probable method of development, the total gross head which it is believed may be developed.

The rating will be shown for the following conditions:

(1) Power based on natural flow available 90 per cent of the time. The 90-per-cent-time-flow is the flow occurring at the site 90 per cent of the time as determined in general by a study of the records of mean daily discharge. In instances where the records are fragmentary or of short

duration, however, estimate of the 90-per-cent-time-flow will be based on all available data and applied over a sufficient length of time to render improbable that additional records will materially change it. Mean weekly or mean monthly flows may give reasonable accuracy but will in general show exaggerated flow.

Theoretical kilowatt = $H \times Q_{90} \times .08$.

Theoretical horsepower = $H \times Q_{90} \times .11$

Where H = gross head in feet corresponding to 90-per-cent-time-flow.

Q_{90} = 90-per-cent-time-flow in second-feet.

.08 and .11 = factors to give approximately 100 per cent efficiency.

(2) Power based on natural flow available 50 per cent of the time; the 50 per cent-time-flow being the flow occurring at the site for half the time based upon a study of the record of mean daily discharge or so estimated that it would not be materially changed were a study to be based on long-time records of daily discharge.

Theoretical kilowatts = $H \times Q_{50} \times .08$.

Theoretical horsepower = $H \times Q_{50} \times .11$.

Where H = gross head in feet corresponding to 50-per-cent-time-flow.

Q_{50} = 50-per-cent-time-flow in second-feet.

.08 and .11 = factors to give approximately 100 per cent efficiency.

(3) Capacity of reservoir sites, above the lowest level of draw-down, expressed either in acre-feet or cubic feet, that is reasonably probable of utilization, directly or indirectly, for increasing the power resources, together with an estimate of the resulting increase in power resource stated in such terms that a comparison may be made with the power resource obtained by rules 1 and 2 of the site or stretches of the river affected by the storage.

Developed sites—The rating of developed sites will show:

(4) Power at the site to be determined in a manner similar to that for an undeveloped site.

(5) Manufacturers ratings of limiting machine capacities expressed in kilowatts.

The sectional committee of the American Engineering Standards Committee is co-operating in this work with the committee of the International Electro-Technical Commission which met in New York last year.

State Aids Housing Construction in Britain

During the eight years which have elapsed since the War, more than 800,000 new houses have been erected in Great Britain, or more than 100,000 a year, and of these the larger part were erected with state aid, according to advices received by the Bankers' Trust Co. of New York from its British information service. The building subsidies are to be reduced on Oct. 1, 1927, so that the peak of construction will have been reached and passed by that date.

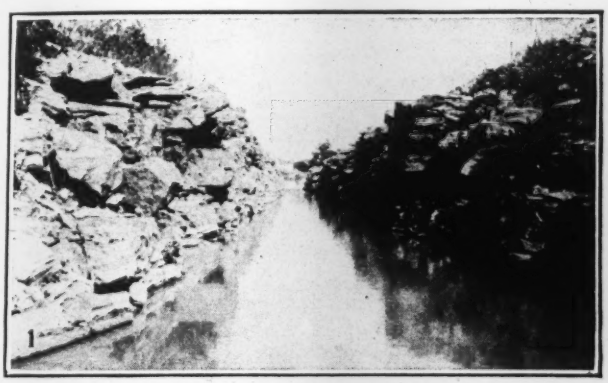
Smoothing Rough Rock Channel to Increase Capacity

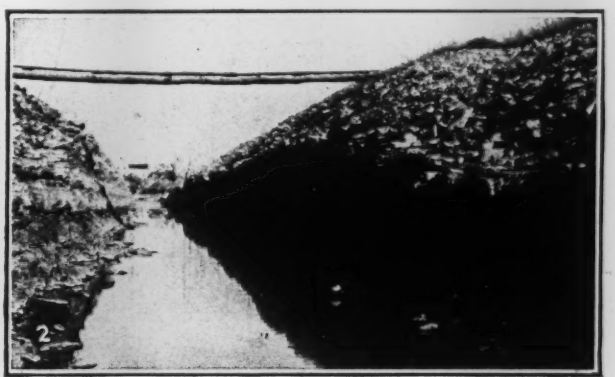
By C. E. Ramser

U. S. Drainage Engineer, Bureau of Public Roads, Washington, D. C.

SINCE the appearance of the article entitled "Kutter's n for Rough Rock Channel Excavated by Explosives," in *Engineering News-Record* of Dec. 6, 1923, p. 936, the channel has been somewhat enlarged and the perimeter smoothed up by hand labor so as to afford a better outlet for a small diversion floodway of the Little River Drainage District in southeast Missouri. An examination of Fig. 1, a view taken after the enlargement, shows large projecting rocks compared with the fairly even and regular surface shown by Fig. 2, taken after the channel sides were smoothed up.

Measurements were made by the writer to determine the value of n in Kutter's formula for these two different conditions of channel, and results are given in the accompanying table. These values for the original channel range from 0.0385 to 0.0420, and for the smoothed-up channel, from 0.0301 to 0.0308, a reduction of n from about 0.040 to about 0.030. Using the same hydraulic elements as were obtained for the highest stage in the original channel and computing discharges for values of n of 0.030 and 0.040, it was found that the smoothing-up process alone would have increased the capacity of the channel about 33 per cent.





FIGS. 1 AND 2—VIEWS OF SALS CREEK ROCK CHANNEL BEFORE AND AFTER SMOOTHING UP
1—As originally blasted out. 2—After sides were smoothed up by hand labor.

EFFECT OF SMOOTHING UP PERIMETER OF A DRAINAGE CHANNEL BLASTED THROUGH ROCK

Hydraulic elements and values of "n" in Kutter's formula
Sals Creek diversion channel, Little River Drainage District, Missouri

Average Max. Depth, Ft.	Average Surface Width, Ft.	Length of Course, Ft.	Slope Discharge, Sec.-Ft.	Average Cross-Section Area, Sq.Ft.	Mean Hydraulic Radius, Ft.	Slope of Water Surface	Mean Velocity, Ft. per Sec.	C in Chezy's Formula	n in Kutter's Formula
Channel as originally constructed by explosives (Fig. 1)									
5.42	10.0	255	160.0	44.1	2.49	0.00268	3.63	44.4	0.0385
5.67	10.1	255	177.9	46.2	2.56	0.00288	3.85	44.8	0.0385
5.61	10.1	161	160.0	46.2	2.56	0.00279	3.46	41.0	0.0420
5.86	10.2	161	177.9	48.4	2.62	0.00274	3.68	43.5	0.0395
Channel as enlarged and sides smoothed up by hand labor (Fig. 2)									
2.50	15.4	250	115.0	35.6	1.93	0.00190	3.23	53.4	0.0301
2.90	15.6	250	146.2	42.0	2.19	0.00190	3.48	53.9	0.0308
3.10	15.8	250	156.9	45.0	2.30	0.00175	3.49	55.0	0.0305

Counterweighted Cables Raise 433-Ft. Spans to High-Level Cantilever Bridge

Two 620-Ton Suspended Spans for Carquinez Highway Bridge Were Floated Under Structure and Lifted by 2½-In. Cables with Counterweights

By Charles F. Goodrich
Assistant Engineer, American Bridge Co., New York City

ERECTION on the large cantilever bridge which is now being completed over Carquinez Strait, Calif., connecting the towns of Crockett and Vallejo, 30 miles north of San Francisco, involved several unusual and spectacular features. The structure comprises two 1,100-ft. cantilever spans, each containing a 433-ft. suspended span. The last two important operations, completed during March, were the hoisting of the two suspended spans from water level to place in the bridge. On March 3 the north suspended span was raised, and on March 19 the south suspended span, completing the last link in the 4,482-ft. structure.

A description of the bridge, together with a map showing the location, appeared in *Engineering News-Record*, Sept. 24, 1925, p. 504. An article on the foundation work appeared in the issue of March 24, 1926, p. 484. [The design of the structure is to be described in an article soon to be published.—Editor.]

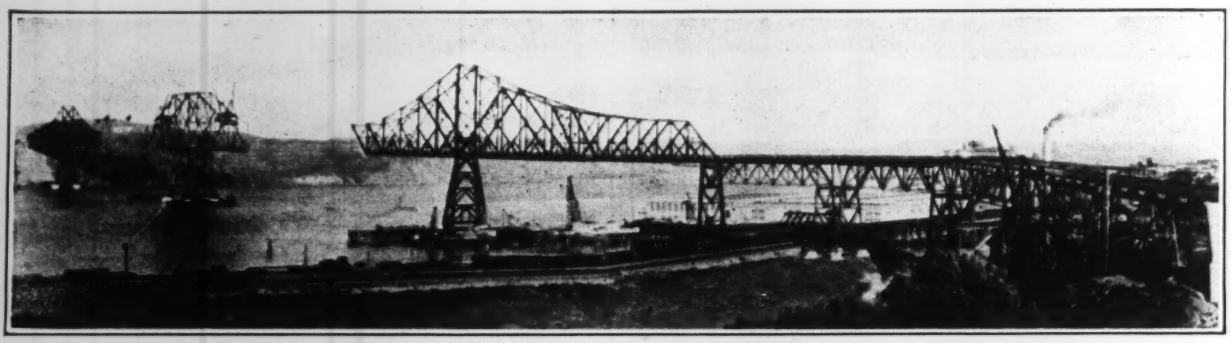


FIG. 1—CARQUINEZ STRAIT BRIDGE NEARING COMPLETION OF CANTILEVER ERECTION (FEB. 1, 1927)

Beginning Erection—The south viaduct approach consists of three 75-ft. deck plate-girder spans and four deck lattice-truss spans on towers about 100 ft. high. Due to difficulties in securing property rights it lies on a 10-deg. curve and passes over ground that is ultimately to be used as a railroad yard. Here falsework was used. Beginning with span 1, erection was carried on by deck travelers working both ways, that is, toward the south abutment and toward pier 5. Then followed work on the two 500-ft. anchor spans, 5 and 13, which was carried on simultaneously but by quite different methods.

South Cantilever—Under the south anchor arm, span 5, the water was shallow, and therefore falsework was used. The deck traveler, proceeding from the viaduct end, placed the floor and all truss members that could be reached, and also the first main tower. It then placed erection floorbeams and stringers just under the top chord in panels 8 to 10 and on them a top traveler. The top traveler then proceeded northward on an upgrade of about 13 per cent, placing all the upper members in span 5 (Fig. 2) and, proceeding thence on a similar downgrade from tower 4, placed all members in the cantilever arm (span 6), raising the steel members from barges below.

North Cantilever—Under span 13, the anchor arm at the north end of the bridge, the water was about 90 ft. deep for about half the length of the span. Here timber falsework was used on the hillside from panel point LO to L8, but at the latter point a temporary steel bent was placed on a temporary pier and about this bent the rest of the span was cantilevered a distance of 277 ft. 10 in. to tower 2 (Fig. 2). This temporary bent was made from steel members afterward used in the erection of the central tower, and carried a maximum load of 5,000,000 lb. The deck traveler placed everything in span 13 up to panel 10 and beyond this a top traveler like that used at the south end, erected the cantilevered portion. Meanwhile a large floating crane of 150-ton capacity obtained from the Mare Island Navy Yard erected tower 2 in time to receive span 13 when ready.

A special arrangement was provided for closing span 13 and connecting the outer end to tower 2. The top of the temporary bent at panel point L8 was equipped with rocker shoes, and this panel point was placed enough higher than its final location to provide vertical clearance between the end of span 13 and tower 2. Span 13 as a whole was erected about 3 in. north of its final position so as to provide horizontal clearance. Temporary eyebars were used at LO to carry the uplift at this point down into the abutments. Closure of this span was made in the following way:

Hydraulic "uplift" jacks at LO were gradually released allowing the uplift to raise this point, rotating the span vertically about point L8, thus lowering point L18 until it landed on tower 2; at the same time the whole span was jacked southward until it could be connected to the tower. As soon as the uplift LO was reduced to zero, hydraulic jacks under the end floorbeam at this point lifted the span from the temporary tower under L8 and, after the rocker shoes had been removed, lowered the span to its final position on pier 1. This operation was performed without any mishaps whatever. The top traveler thereupon proceeded on its way

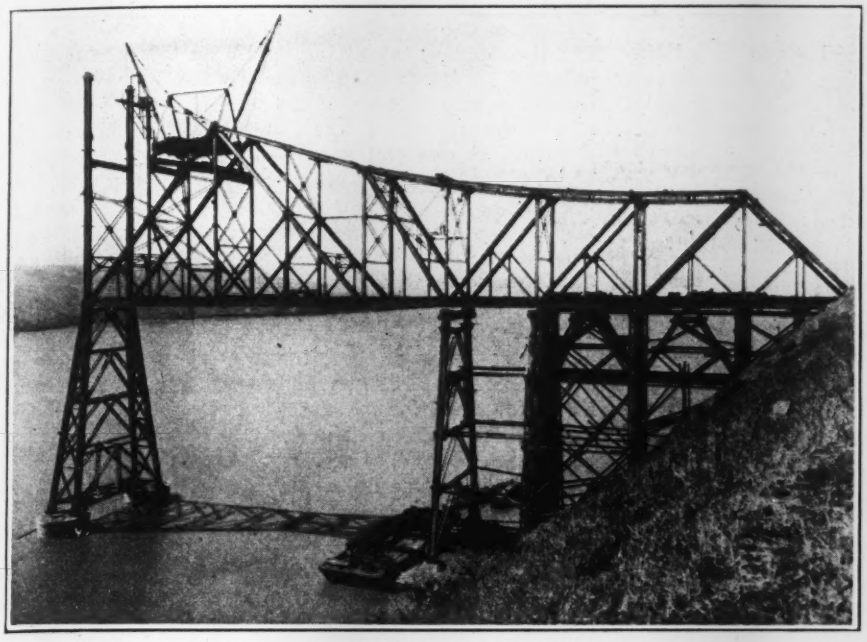


FIG. 2—NORTH ANCHOR ARM ERECTED BY CANTILEVERING OVER TEMPORARY BENT
Traveler on inclined track below top chord handled outer half of work.

southward and erected the cantilever arm (span 12).

Central Cantilever—While spans 6 and 12 were being erected the Navy Yard crane placed steel in the central tower (span 9) up as far as the floor. As soon as the top travelers in spans 6 and 12 could be released they were taken down and placed in tower span 9, where they proceeded to erect the upper portion of span 9 and then, working in opposite directions, to erect the two central cantilever arms (spans 8 and 10).

All steel placed over the water was delivered to the various points by two steel barges which were loaded from a dock built on piles out into the stream and connected by track to the material yard. This dock also served as falsework for the two suspended spans, erected side by side during placing of spans 8 and 10.

As soon as the north arm of the central cantilever, span 10, was finished its top traveler was removed, but the traveler in span 8 was left at its outer end to aid in counterbalancing the north suspended span (span 11) and its counterweight during raising.

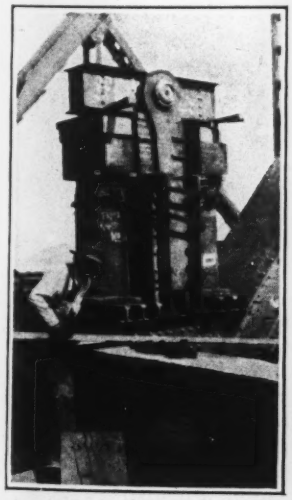


FIG. 3—UPLIFT JACKS AT NORTH ABUTMENT

Hoisting the Spans— A rather novel method of raising the two suspended spans (7 and 11) was adopted. The preparation of the cantilever arms to receive one of these spans was:

Two wooden boxes about 13 x 14 x 19 ft., one for each truss, were hung from panel points M29 on each cantilever span, and filled with sand to give a weight slightly in excess of the suspended span weight. To each box were connected, through an equalizing device, two monitor plow steel wire rope cables, 2½ in. in diameter, which passed up over sheaves 5 ft. in. diameter at points M29, over other similar sheaves at points U30, and then hung free with the ends socketed to give the proper length to connect to the ends of the suspended span when floated into position. These cables were 250 ft. long; their free ends hung a few feet above water level.

Hoisting the North Span— On the morning of March 3, at approximately low tide, the two 40 x 130-ft steel barges, fabricated especially for this job, were warped under suspended span 11 at the dock, one barge at each end, and centered midway between the second two panel points, transversely to the span. The span weight was then transferred by means of eight 500-ton hydraulic jacks from the falsework to the barges, the jacks being placed under the panel points and the follow-up blocking under the four floorbeams. The span was then further jacked up to a height of 11 ft. between the top of the barges and the underside of the span. This height was to provide for lost distance at the time of transferring to the raising cables, due to buoyancy, deflection of the cantilever arms, elongation of main cables and heights of jacks. The span was guided and held by 12 x 12-in. wooden brackets bolted to the decks of the barges.

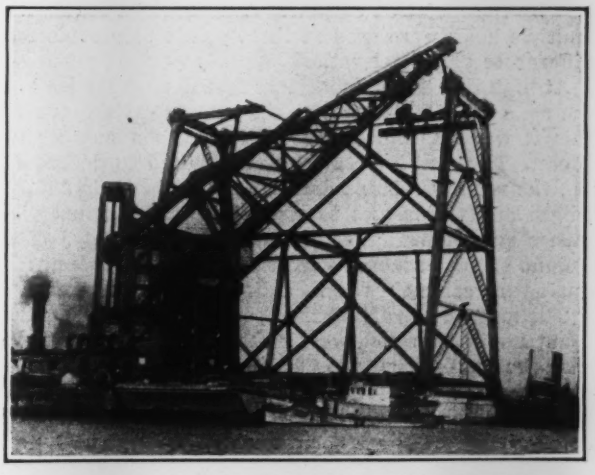


FIG. 4—NAVY CRANE ERECTING CENTRAL TOWER
Floating crane of 150 tons capacity reaches 160 ft. to place 30-ton members.

As soon as the jacking operations were completed four large tugs drew the barges and span out into the stream, where it was allowed to float with the incoming tide through the south channel, moved over to the north channel, and towed into position against the tide. Here

lines were put out to four anchors, previously placed, to center the span transversely, and lines to the adjoining piers to center it longitudinally. As soon as this had been done and the tugs freed, the raising cables were attached to the four corners of the span at points

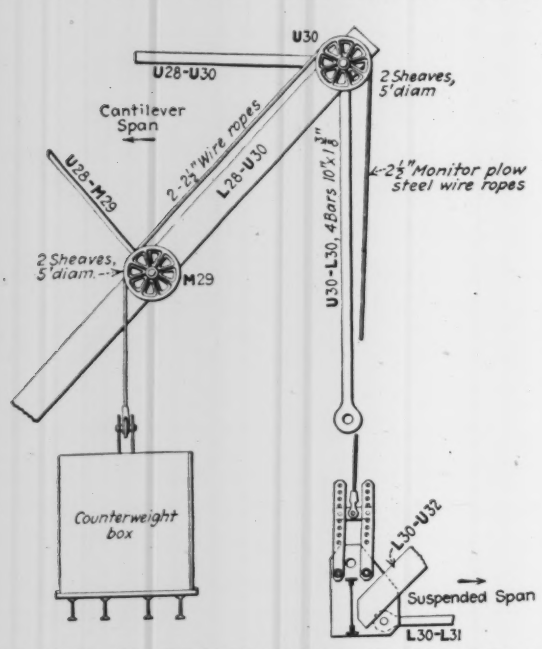


FIG. 5—ROPE AND COUNTERWEIGHT ARRANGEMENT FOR HOISTING SUSPENDED SPAN

L30 by the insertion of pins. Long upright bars with multiple holes were used at these points to provide for differences of tide elevations.

At high tide the 500-ton jacks transferred the load of the span from the barges to the raising cables. To do this it was necessary to float the jacks a number of times. The cantilever arms were figured to deflect 6½ in., the raising cables to stretch 18 in. in their 250-ft. length and the barges to rise 21 in. The actual distances agreed with the calculations very closely. The weight of the suspended span (stringers were omitted during the raising) was, including rigging, 620 tons.

The method of raising the span was by lowering the counterweights, which weighed 22 tons per corner more than the span. The excess weight was provided in order to overcome all friction and inertia and to overhaul the hoisting engines, plus a slight excess for a factor of safety. Lowering the counterweights was done by an eight-part ¾-in. wire rope falls to each counterweight box, the two lead lines at one end of the span being slacked off simultaneously by the two drums of a 55-hp. electric hoisting engine. Before lowering the counterweights their excess weights had to be picked up, one corner at a time, in order to remove the supporting pins. Then at all four corners the lowering of the counterweights was begun, raising the span.

In order not to overtsress the bracing system, due to uneven raising of the four corners, it was quite necessary to keep any warping of the span within a reasonable limit. Scale marks were painted near the ends of each end floorbeam at equal heights; they were observed through transits, one for each end of the span, set up on a horizontal strut halfway up the adjacent towers. The maximum difference in level between the two corners at one end of the span did not exceed 2 in. at any time during the lifting, and even this was largely balanced, so far as warp was concerned, by corresponding differences in levels of the corners on the other end of the span; that is, when there was a difference in elevation at one end approaching 2 in., there was a 1½-in. difference at the other end, the lower corners being on the same truss. Thus there was only a very slight warp in the span at any time. It was not considered necessary to level up for this small difference, but it was done in order to be sure that the method of doing it would be effective in case of a further deviation at a later stage of the operations.

It was necessary to keep the two ends of the span only approximately at the same level. Observance of the level, however, was made possible by indicator boards painted like stadia rods and mounted on the cantilever arms, each being observed through binoculars from the opposite arm. The difference in level between the two arms was at one time about 8 ft., which could have been exceeded considerably without causing any difficulty.

All operations were controlled by telephone from a central station located on the deck at the outer end of one cantilever arm, from which communication was maintained with the opposite cantilever arm and with the two transitmen on the towers. All three telephone lines were brought into a single receiver at the control station, so that communication was not subject to the delay of switching. As soon as reports were received from the transitmen as to levels, the signals were given for lowering simultaneously at each end. A number of stops were made on the way in order to observe levels and to make sure that there was no fouling of tackle, etc.

The actual time from the first movement until the span was in position was about 40 min.

Before the span was lifted off the barges block and tackle lines had been attached to the hip points of the span and their lead lines taken to the niggerheads on the operating engines above as an emergency rig. These

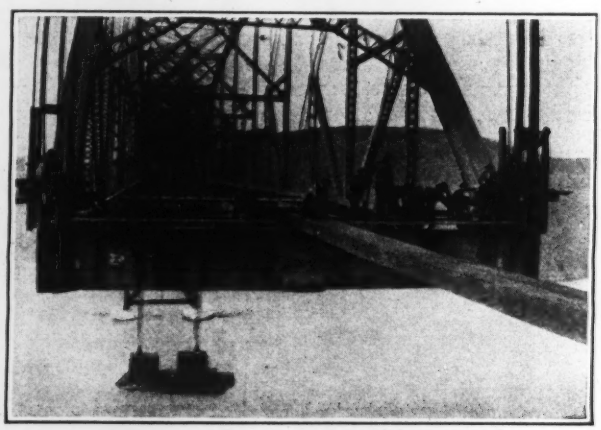
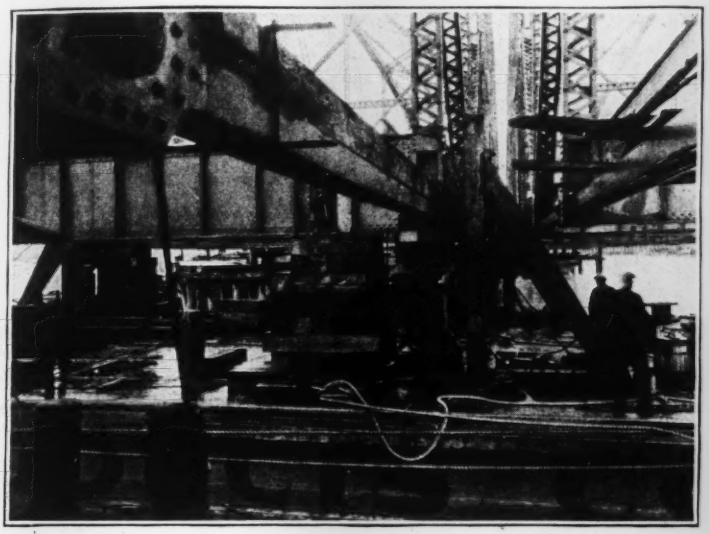
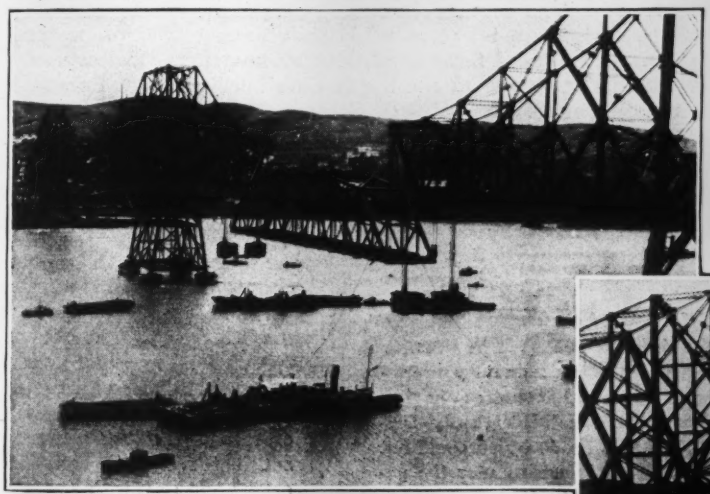


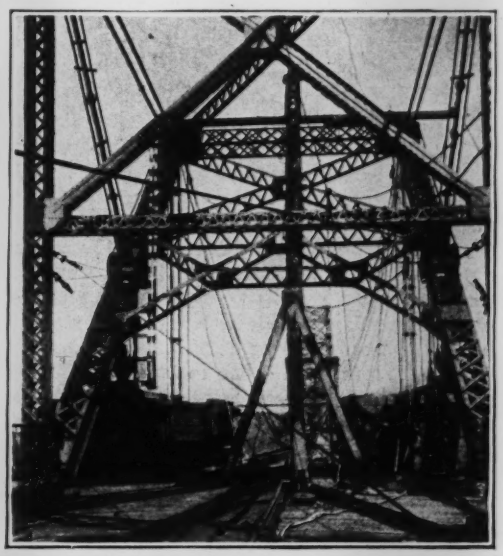
FIG. 6—DRIVING HANGER PIN AT END OF SUSPENDED SPAN

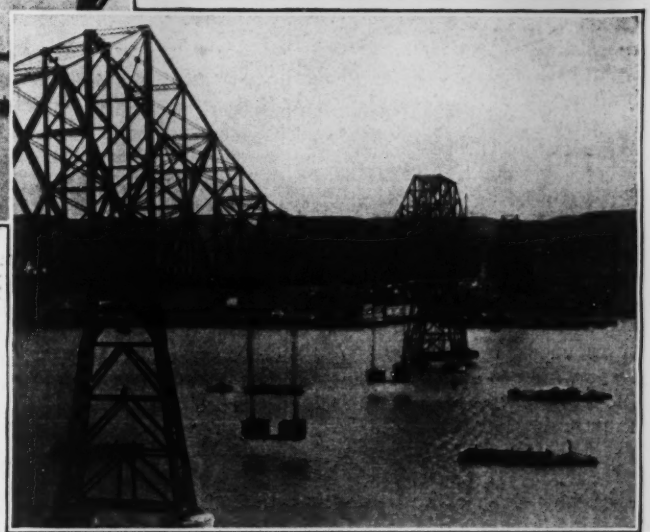
lines were crossed, to aid in preventing any swaying of the span, and slack was taken up as the lifting progressed. It was found unnecessary to use them during the raising of the span, but as time did not permit of placing the regular lateral system until next day they performed the function of providing a temporary lateral system between the cantilever arms and the suspended span after the latter had been raised.

As soon as all four pins were driven at points L30, trap doors in the bottoms of the counterweight boxes were opened and the sand was allowed to flow out onto the barges to be used again later in filling the boxes for raising the south suspended span.









FIGS. 7 TO 10—HOISTING ONE OF THE SUSPENDED SPANS OF CARQUINEZ STRAIT BRIDGE

Upper left—Four jacks at each end transfer span from falsework to scows; timber brackets hold span against shifting.

Upper right—Counterweights hanging at end of cantilever arm ready to take weight of span; gage board at left.

Lower left—Span being hoisted; note crossed emergency lines to hip points. Total hoisting time was 40 min.

Lower right—Suspended span in position and connected; note transitman in tower at left.

South Span Hoisting—Span 7 was raised in the same manner on March 19. The time consumed in the actual raising of this span was 35 min. for the lift of about 125 ft. All operations on each span were completed in one day.

The Carquinez bridge was erected for the American Toll Bridge Co., for which Charles Derleth, Jr., as chief engineer, has been in charge of the design and construction of foundations and superstructure. Under his supervision the design and contract drawings for the steelwork were made by D. B. Steinman of the firm of Robinson & Steinman, consulting engineers, New York. To adapt the work to shop practices and erection necessities, modifications in the design were made and erection schemes and programs worked up by the American Bridge Co. in New York in the office of C. W. Bryan, chief engineer, under supervision of C. G. E. Larsson, assistant chief engineer, and in charge of C. F. Goodrich, designing engineer, subject to the approval of Wm. H. Burr, consulting engineer, in addition to Messrs. Derleth and Steinman.

The steelwork was erected by the United States Steel Products Co., Pacific Coast representatives of the American Bridge Co., under the supervision of Richard Khuen, Jr., general manager of erection, and K. L. Strickland, division erection manager. In the field Frank Stangle was in charge as foreman and Allen F. McLane was resident engineer on the Carquinez Strait bridge.

State Highway Developments of 1926 Awarded Blue Ribbon of Distinction by State Highway Engineers

FIFTH ARTICLE

Six Counties Combine to Build 155-Mile Coastal Highway

BY F. M. GARNETT

Office Engineer, State Highway Department, Savannah, Ga.

COASTAL HIGHWAY IN GEORGIA

GEORGIA is building highways on a pay-as-you-go basis, a system which makes it difficult for poor counties to do much in the way of building major roads; and most of the coastal counties in Georgia are very poor. Outside of the cities of Savannah and Brunswick, there is little wealth, the rural sections never having recovered from the destruction of civil war days which resulted in the rice fields reverting to swamps and the corn and cotton fields to forest land. Therefore, in order to build the important trunk line described here, the counties of Chatham, Bryan, Liberty, McIntosh, Glynn, and Camden in 1924 organized the Coastal Highway District. The district was granted corporate status by an act of the Georgia state legislature and permission to issue bonds was given in an almost unanimous vote of the people of the state.

An estimate of the cost of building the highway was placed at $3,600,000 by the engineers of the state highway department. This included building an entirely new highway for a large part of the total distance of 155 miles from the South Carolina to the Florida line, and the reconstruction of portions of older county highways which could be included in the new route. One-fourth of the cost is to be derived from a bond issue of $900,000 by the Coastal Highway District and the other three-fourths is to be obtained from state and federal aid funds. The cost of carrying the $900,000 bond issue of the highway district has been apportioned among the counties on the basis of taxable values rather than on the mileage of the new highway in the counties. Thus, Chatham County, in which Savannah is located and where the major portion of the coastal highway will follow routes that are already hard-surfaced, will pay about 76 per cent of the cost, and Glynn County, in which the port of Brunswick is located, will pay 13 per cent. On this basis, the two cities will meet nearly 89 per cent of the whole cost of the bond issue, but they expect to be amply repaid by the increased development of the rural communities which will be made accessible by the improved road and the through traffic which will follow the new route.

GEORGIA

CONSTRUCTION of the coastal highway along the Atlantic seaboard in Georgia is being carried on as a community undertaking, the six coastal counties having united to form the Coastal Highway District in order to raise funds which could not have been obtained otherwise under the peculiar local conditions affecting such work. The highway is needed not only to open up these lands for new developments which are now being undertaken in order to restore that fertile region to a prosperous condition, but also as a link in a chain of highways running the full length of the Atlantic seaboard.
—W. R. NEEL, *State Highway Engineer.*

The country through which the highway runs is a low coastal plain made up of many swamps, marshes and sand flats, deeply cut by numerous tidal estuaries. Beginning at the South Carolina line above Savannah, where the states of South Carolina and Georgia have recently completed a 4-mile crossing of the Savannah River and its delta, the highway runs down through Chatham County into Savannah and then turns back inland again to a crossing over the Ogeechee River at the county line. In Chatham County it follows existing hard-surfaced and concrete roads, except for a short piece of new road to reach the Savannah River crossing. From the Ogeechee River crossing, the route passes south through Bryan, Liberty and McIntosh counties over a location that is partly along the line of the old county roads and partly an entirely new highway as far as Darien. The grading on this portion is completed and contracts have been let for all of the surfacing, 11 miles of asphalt, and 33 miles of concrete.

From Darien to Brunswick, crossing the Altamaha River delta, the route follows an existing hard-surfaced road into Brunswick, which is down on a peninsula between the sea coast and the Turtle River.

Beyond Brunswick the existing road runs northwest for several miles in order to get inland to cross the Turtle and Satilla rivers and thence runs south again roughly parallel to the Seaboard Air Line Ry., but with many meanderings to avoid swamps and to reach satisfactory river crossings. The new highway follows an entirely different route. Leaving the present

highway at a point far enough above Brunswick to avoid the greater part of the navigation of the port, it crosses the Turtle River on a bridge 2,940 ft. long to reach an island between the Turtle and South Brunswick rivers and continues in a southwesterly direction across the South Brunswick River and the Little Satilla River until it reaches the Seaboard Air Line Ry. at

TYPICAL SURFACE TREATMENT NEAR SAVANNAH

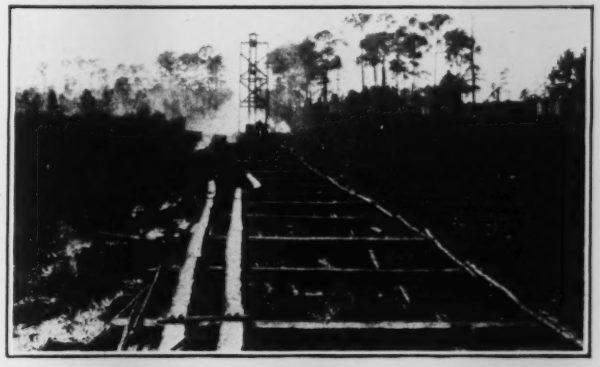
Waverly. From this point it runs almost due south parallel to and on the east side of the railway to the Florida line. Only 3½ miles of this section will follow existing highways. By keeping to the east of the railway and crossing the rivers closer to Brunswick, 15 miles of distance is saved and 13 grade crossings are eliminated.

The location of the line between the Little Satilla River and Waverly is not approved by the Bureau of Public Roads, the bureau favoring a more direct line to the Satilla River, and as a consequence the district is building this portion without federal aid.

The portion of the highway from Brunswick through Glynn and Camden counties to the crossing of the St. Marys River at the Florida line is all under contract and in various stages of completion from clearing the right-of-way to a completed grade ready for hard surfacing.

Construction Details—Grading work is practically all side borrow or filling from low trestles, the side borrow being hand or dragline work depending upon the character of the surrounding country and the amount of fill required. Sand is used wherever it is available, but in many places it is too far away to be hauled in economically except for making the top of the fill. In such cases the embankment is built up with black muck from the surrounding swamp land by dragline excavators if the fill is of considerable height and if material close to the center line can be used to advantage, or by hand work from borrow pits if the fill is low or if better material can be obtained by going to a greater distance from the center line. Wherever it is possible to do so, an 8-ft. berm is left between the borrow pit and the toe of the fill. Contractors are paid on neat sections plus 15 per cent for shrinkage on ordinary fill work, but they usually put up an excess of material into the fill in dragline work in order to be on the safe side, as dragline material costs from 10 to 12c. per cubic yard in place, while material hauled in to make up deficiencies after the dragline has moved on costs about 40c. a yard. On fills across marshes payment is made by borrow pit measurement. Considerable care is taken in grading the borrow pits and in connecting them with waterways so that they will drain properly with the ebb and flow of the tide.

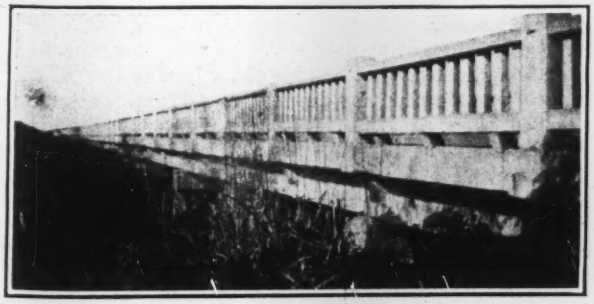
Selected material to a minimum thickness of 8 in. in



MAKING FILL IN MARSH LAND

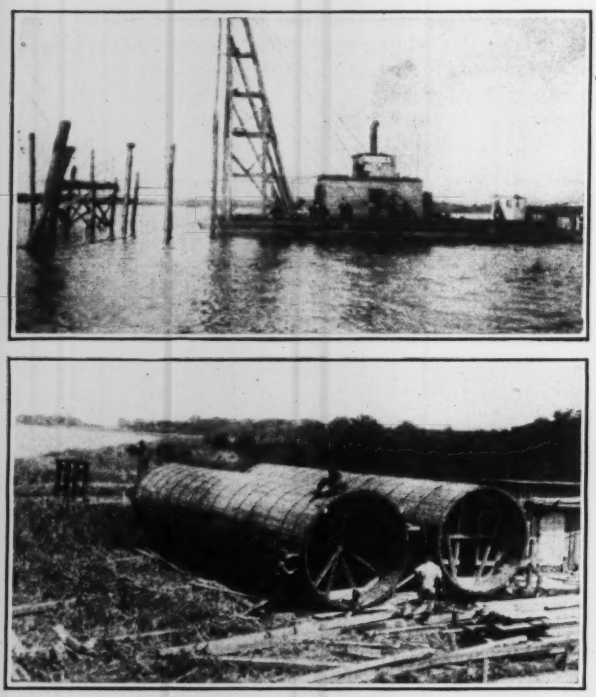
Above—Temporary trestle with stringers for track at left side. Below—Delivering sand to fill with gasoline engine and dump cars.

the center is hauled onto the top of the muck fills from borrow pits along the right-of-way. Providing enough of this material of a satisfactory character is one of the chief problems, as the sand at best is so fine that it makes a poor road surfacing material. It is all right when wet, but as soon as it dries out it is so light that wheels cut into it deeply. As a result, the better material is hauled for long distances. This top surface on the muck fills is usually made 12 in. thick to allow for irregularities in the shrinkage of the fills, or for possible subsidence. Subsidence, however, up to the present time has not been serious except in three places, the north approach of the Sapelo River, the south approach of the Satilla River bridge, and the north approach of the St. Marys River bridge. At the Satilla River extra bents were added to the bridge approach to avoid further filling on the soft ground.



CONCRETE TRESTLE ACROSS SAVANNAH RIVER DELTA

The fill for the St. Marys River approach was made by sand dumped from an industrial railway on a temporary trestle by methods similar to those used in making railway fills, the stringers being removed after the fill reached the deck so that the remainder of the material can be placed from track laid directly on the fill to aid in compacting it. During this work, considerable settlement occurred near the river and the marsh was raised up on both sides to several feet above its original level. Recent observations seem to indi-



TYPICAL BRIDGE WORK

Above—Driving the first piles for the center span of the Turtle River bridge. Below—Timber caissons for Satilla River bridge. Bents of approach trestle in the background.

cate that the subsidence of this fill has practically ceased. Further slight subsidence will be of little consequence as this portion of the road is to have a temporary surfacing of gravel which can be added to, if necessary, before hard surfacing is undertaken.

Embankments are built with a subgrade width of 28 ft. and a finished roadway of 18 ft. Both the shoulder and embankments are sown with Bermuda grass to form a thick sod which will protect the fine material from scour during the semi-tropical rains to which the region is subject.

Bridges—The Savannah River crossing is made up of a series of reinforced-concrete trestles and concrete girder spans on concrete piles with one 240-ft. swing span at the main channel crossing. It was described in *Engineering News-Record*, Oct. 8, 1925, p. 590. The bridge over the Ogeechee River is of similar construction, precast concrete pile trestle approaches and a steel swing span over the channel. South of Brunswick the bridges are largely of creosoted timber construction, timber trestles where no channel openings are required and steel trusses on concrete piers over the wider channels where swing spans were required. The bridge piers are either pairs of 10-ft. diameter concrete cylinders for fixed spans or large cylinders for the center piers of swing spans, all carried well into a hard bed of marl which underlies the muck and mud in the rivers and marshes.

The creosoted timber structures are built of material obtained from local creosoting plants.

Between the Ogeechee River and Darien the bridges are constructed of black or red cypress secured locally.

The St. Marys River crossing was built by the Florida State Highway Department according to its own plans. It consists of a steel swing span over the channel flanked by steel trusses over the side channels and approaches of short girder spans of concrete, the total cost being $260,000.

Work on the Turtle River crossing, consisting of four bridges totaling 4,449 ft. over deep water, the largest of the remaining bridge jobs, is well under way. Completion of these structures, which are the controlling link in the southern end of the work, is set for October, 1927. Road work south of that point is

LOOKING NORTH FROM THE ST. MARYS RIVER BRIDGE

The embankment in this section was made of sand from a borrow pit beyond the swamp. Considerable subsidence occurred in the portion close to the river.

timed to be completed on or before the date of the completion of the bridge, so that the highway can be opened for its entire length before next winter.

Organization—Construction work is being carried on by the State Highway Department in co-operation with the Coastal Highway District. In this way, the highway department is not hampered by having to deal with each of the six counties directly, as all local matters are handled through E. L. Stephens, field superintendent for the highway district. W. R. Neel is state highway engineer and Searcy B. Slack is bridge engineer. Direct supervision of the work is in charge of the division office at Savannah, where H. J. Friedman is division engineer, E. A. Logan, field construction engineer, and the writer, office engineer. Harvey Granger, of Savannah, is chairman of the Coastal Highway District.

Flow Tests of Winnipeg Water District Aqueduct

Salt Velocity Method Proves Quick and Accurate and Checks with Current Meter Tests After Six Years' Use

By F. S. Adamson and J. H. Grant
Engineers, Concrete Steel Co., Akron, Ohio

In the fall of 1925 a series of flow tests were made on the aqueduct of the Greater Winnipeg Water District to determine the effect of six years' operation on the carrying capacity. After carefully considering all methods of flow measurement, the salt velocity method, developed several years ago by Professor Allen for testing turbines, was chosen as one that could successfully be employed on this type of work, and that would give a new and hitherto unattained degree of accuracy.

It was at once apparent that some factors were uncertain, and it was necessary to do considerable experimenting to determine the actual value of these unknown quantities. No data were available as to the correct saline density of the solution to be used in order to produce sufficient variation on the recording instruments, or as to the voltage capacity required to record the passage of the salt solution. There was also a question as to what size of electrodes to use, and the spacing that would be consistent with the other two unknown factors.

The aqueduct is concrete, with cross-sections varying from 5 to 9 ft. across. The tests were made in sections of the aqueduct near the intake end, about 84 and 94 miles from Winnipeg. Two different cross-sections were used, both being horseshoe-shaped and having inside heights of 7½ and 9 ft. respectively. The concrete was

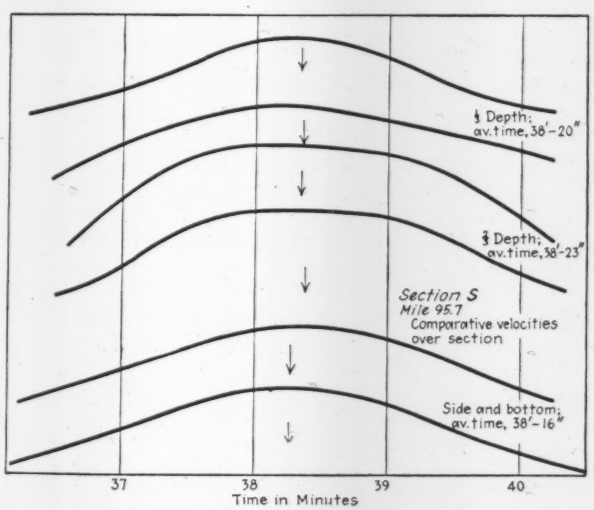


FIG. 1—COMPARATIVE VELOCITIES OVER TEST SECTION OF WINNIPEG AQUEDUCT
Taken ⅓ depth, ⅔ depth and bottom.

exceptionally good, every precaution having been taken to secure proper proportioning and spacing. A colorimetric test made in 1919, on completion of the work, and before the water was used for consumption, showed that while the aqueduct was designed to give a maximum flow of 91 Imp. m.g.d., the actual capacity was 111 Imp. m.g.d. This test also showed that although a design value of $n = 0.013$ in Kutter's formula had been used, the actual value obtained was $n = 0.0115$.

After six years' operation, the capacity of the aqueduct, as shown by the tests here described, was found to be 100 Imp. m.g.d., which is still 9 Imp. m.g.d. above the design capacity. The value of n for the test flow

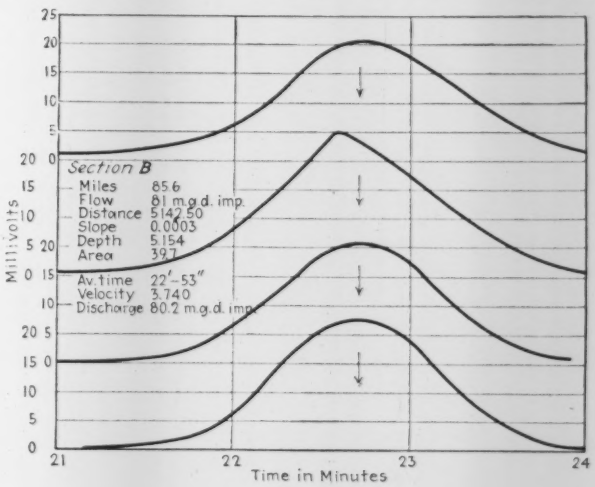


FIG. 2—MILLIVOLT READINGS FOR SET OF FOUR RUNS FOR 80-M.G.D. FLOW, WINNIPEG AQUEDUCT

of 81 Imp. m.g.d. was found to be 0.012 which is still below the actual design value of $n = 0.013$ computed for maximum theoretic flow of 91 m.g.d.

Conditions for carrying out the tests on this aqueduct are very favorable. The total length is approximately 100 miles, with long tangents, laid to a very accurate grade. Two lines of precise levels had been run on the finished invert. From the resulting profiles, the two sections used were chosen, such that uniform grade was secured for long distances. Backwater effect was avoided, since the test sections were a considerable distance from any changes of grade or section. Another very important feature was the uniform regulation of the flow. Located as these sections were, it was possible to adjust the control gates for any desired flow, and to maintain that flow with little or no variation for any period of time. The flow was always regulated 24 hr. before tests were made, to remove any error that might result from a change in flow during a test period. Manholes were spaced approximately one mile apart and tests were made on two consecutive miles of each section, as a check on the results.

Due to the uncertainty in connection with the first series of tests, a set of current meter readings was taken to act as a check on the method until it was sure that results obtained were correct in every respect. A third check was obtained from a Venturi meter, built into and comprising part of the line.

The current meter was operated from a staging built over the water at an enlarged section of the aqueduct. Due to the horseshoe-shaped section, it was very difficult to operate the current meter so as to get the results desired. It might be added that the temperature was well below zero during all the tests, and caused much inconvenience to the current meter operators, while little or no inconvenience was encountered in the operation of the salt velocity method.

The equipment was chosen and assembled with the idea of ease in transportation and adaptability to field

conditions. It consisted of two 6-volt storage batteries, one millivolt, one voltmeter, electrodes, one 13-quart pail, sufficient cable to connect the apparatus, and salt in 50-lb. sacks. The electrodes used were pieces of ordinary galvanized iron, fastened to a pole and spaced about an inch apart. The most suitable size was found to be 6x12 in. The electrodes were placed at varying positions on the pole, so that with the pole resting on the bottom, the plates were at a known depth. Experiment showed that variation between maximum and minimum rates, due to the position of the plates in the cross-sections, was very small, and followed no accepted ideas. The variation was probably due to errors in manipulation and was negligible. A series of tests revealed that electrodes placed about half way down gave as close results as were consistent with other factors. Several of these tests are shown by Fig. 1.

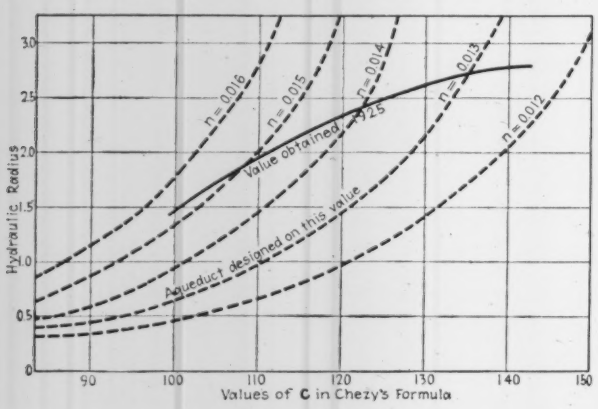


FIG. 3—VARIATION OF "C" IN WINNIPEG AQUEDUCT TESTS

The salt solution finally used was about 5½ lb. of salt to 10 quarts of water, as thoroughly dissolved as possible. This solution was dumped into the stream and the time recorded. Timing was done at first by stop watches, but experience showed that the best grades of these were inaccurate over long periods, so pocket watches of good make with second hands were finally adopted. Checks as to operation were provided at regular intervals to eliminate gain or loss.

Four runs were made for each section and for each flow to investigate the method thoroughly. With good watches and mechanical timing, the results secured show that but two runs are necessary for a test, and that these can be depended upon to check consistently, as long as conditions are steady. Since two salt solutions may be inserted at 10-min. intervals, this method provides less chance for changing conditions than others.

Readings were taken on the millivoltmeter at constant intervals, and then plotted to a time base. The center of gravity of the area under the curve gave the average time of the charge passing the electrode. No matter what shape the curve assumed, it was found that the center of gravity for each run checked very closely (Fig. 2). For swift currents a charge of salt required 35 min. to traverse a mile, while for the slow current the time elapse was 68 min. For the low flow—14 m.g.d. —the current meter was useless, but very satisfactory checks were obtained with the salt velocity method. Only one series was run on this flow, since the time required to study the flow used up all the time allowance at the disposal of the party, due to consumption demands. The Venturi meter gave very unsatisfactory results at this low flow, considerable manipulation being required to secure a reading at all within reason.

The time required for a charge to pass an electrode varied from 4 to 9 min., depending on the flow. The millivoltmeter returned each time to its previous steady reading, and proved very sensitive to a rise in conductivity. Salt in suspension "tailed" out the curve in some instances.

The method of operation was to open a manhole, insert four charges of salt at constant intervals, and proceed to the next lower manhole. The apparatus could be set up and readings then recorded as the charge came down. By this means, the two sections ten miles apart were covered in a half day. It would be very easy to cover long pipe lines thoroughly in short periods of time and so procure very complete records of the condition of the line.

This series of tests, lasting altogether for six days, spaced intermittently over a month, was not at all exhaustive and was largely experimental. Sufficient data were secured with the apparatus used to form certain definite conclusions:

(1) The salt velocity method can be adapted to aqueduct conditions with sections one mile long. Notwithstanding the long period of time required to traverse the pipe, salt injections remain of easily measurable density. These injections give constant results for any one section and flow.

(2) The center of area of the time voltage curves is constant, regardless of the shape of the curves, for the same section and flow. Time values checking within 1 to 3 sec. in 45 min. are not unusual.

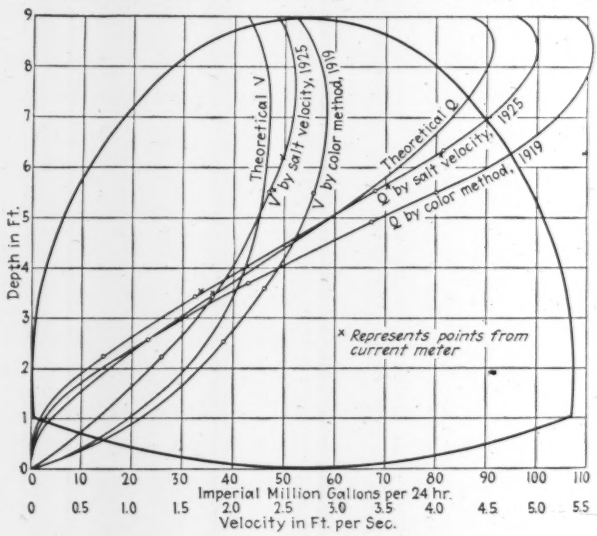


FIG. 4—CAPACITY AND VELOCITY CURVES, SHOWN BY WINNIPEG AQUEDUCT TESTS
Capacities are in Imperial gallons.

(3) The method is positive, giving very constant results. These results check very closely with current meter results, and have, to all appearances, the added attraction of greater accuracy. It is recognized that refinement of injection, recording and timing would decrease the error.

(4) This method is to be commended for its ease, rapidity and low cost of operation. It is applicable where the current meter fails, and certainly is more accurate under low heads.

(5) An interesting feature is the shape of the curve of C in Chezy's formula (Fig. 3), when platted with hydraulic radii. Design basis follows the curve marked $n = 0.013$, whereas, the actual curve is as shown.

(6) $n = 0.013$ in Kutter's formula appears to satisfy safe design, for the value of n is still lower than that figured after five years' operation.

(7) Fig. 4 shows the quantity and velocity curves for design, and the actual curves obtained. It will be noted that current meter results check those obtained by the salt velocity method very closely.

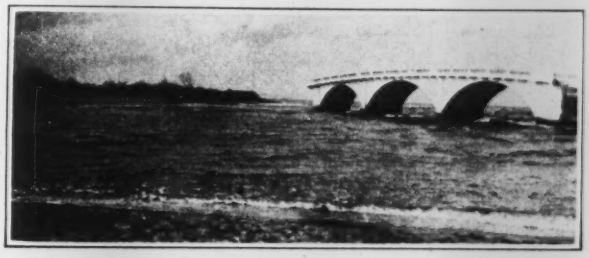
The work was carried out through the kindness of and assistance offered by the Greater Winnipeg Water District, through Commissioner W. M. Scott, and through Prof. J. N. Finlayson, of the civil engineering department of the University of Manitoba, to whom acknowledgments are made. Associated with the writers in carrying out the tests were F. T. Robertson, J. A. McFadden and J. M. Kennedy.

Steel Sheetpiling Used in Repair of Bridge Approaches

Heavy Sections Form Walls of Sand Filled Coffers Strengthened by Tie-Rods and Channels —Florida Structure

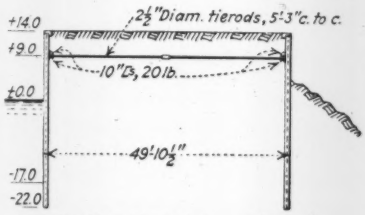
DURING the Florida hurricane of last fall an inlet just north of Miami Beach which connects Biscayne Bay and the ocean was so enlarged at its ocean end that the abutments of a three-arch span concrete bridge crossing the inlet and connecting Miami Beach with Hollywood, Fort Lauderdale, etc., were washed out. The inlet had been cut to create a current in the bay at its north end. As there was no protection to the sand banks on which ran the road leading to the bridge a long stretch at each end of the bridge was eroded, and the heavy seas left an inlet about 1,500 ft. wide. The bridge itself was not appreciably moved.

The bridge had originally a short groin made of rock-filled steel sheetpiling running oceanward at the abutment, but when the approaches were washed away the groin was destroyed and the rock scattered. As valuable real estate abutting the bridge on the land side is now under water, the necessity arose to reconstruct the approaches and confine the inlet to its original banks as soon as possible.

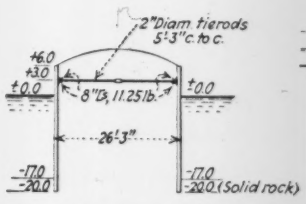


BRIDGE APPROACHES DESTROYED BY HURRICANE

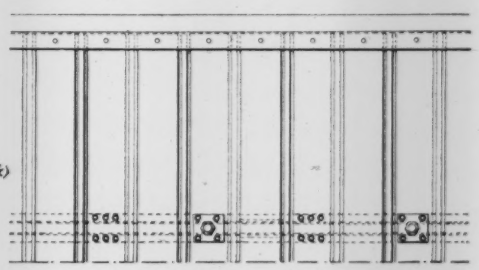
To this end the Bakers Haulover Bridge Commission, acting for the county, accepted plans for rebuilding the approaches. The reconstruction work consists essentially in the erection of parallel walls of steel sheetpiling 50 ft. apart, tied together at proper intervals with 2½-in. tie-rods and the space between walls sand-filled; and the



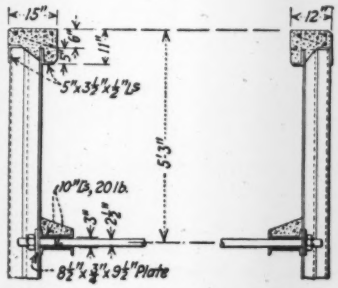
Section Through Approach



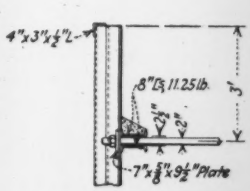
Section Through Groin



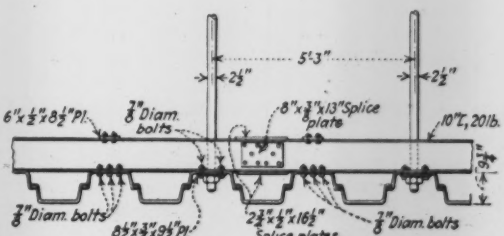
Elevation of Wall and Joint of Channels



Waling and Capping Section



Section Showing Guard Angle and Waling of Groins

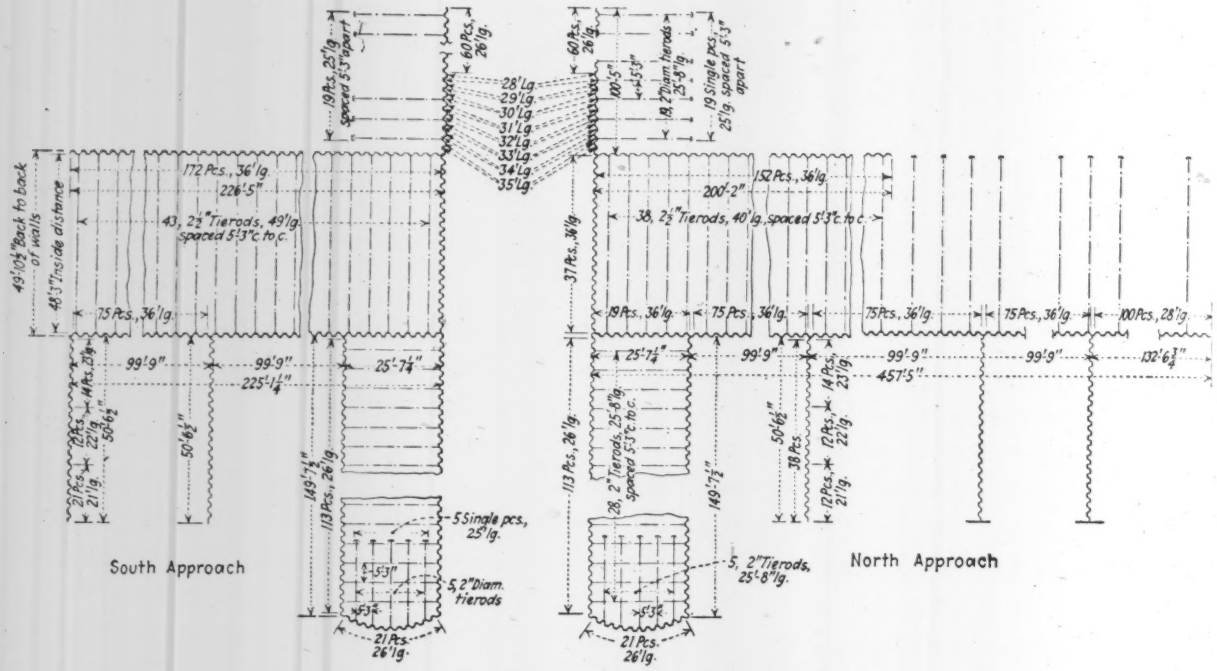


PLAN AND DETAILS OF BRIDGE APPROACH

driving of 150-ft. groins 25 ft. wide, the walls to be tied with 2-in. tie-rods and the space sand-filled, at each abutment; and the erection of 50-ft. groins paralleling the 150-ft. ones at 100 ft. intervals each side of the abutments. Wing walls will extend inward from the bridge for at least 100 ft. to define the channel on the land side.

450 ft. and the inner wall approximately 200 ft. Due to the current and prevailing winds the tendency would seem to be to cut to the south of the bridge and make to the north of the bridge.

The work will go forward under the supervision of Hobart Crabtree, county engineer. J. C. Baile is chairman of Bakers Haulover Commission.



RECONSTRUCTION WITH STEEL SHEETPILING

Rock is encountered at El. —16 to —17 and the road level is approximately 14 ft. above low water. Piling in 36-ft. lengths is to be driven and tests have shown that satisfactory penetration of from 2 to 3 ft. can be obtained in the hard limestone coral, known locally as flint rock, and unlimited penetration in the soft coral.

At the south approach the wall on the ocean front will be constructed of steel sheetpiling having a section modulus of 25.35 in. per foot, extending back 425 ft. from the bridge. The balance of the wall will be constructed of piling with a section modulus of 15.8 in. per foot. Two 10-in. 20-lb. channels will be used as wales between which will run the 50-ft. 2½-in. tie-rods on 5 ft. 3 in. centers. These rods will be equipped with turnbuckles in the center.

A recent survey showed that the length of the ocean piling wall at the south approach would have to be 750 ft. with the possibility that this may be extended if any appreciable amount of erosion takes place before piling can be driven. The inner wall is designed to extend 400 ft. back from the bridge, beyond which single 25-ft. piles will be used as anchor piles on a spacing of 7 ft. 10½ in.

Work will first proceed by driving a line of piling from the water's edge to within 100 ft. of the bridge to cut off the inside flow at high tide, after which driving will progress in both directions as fast as material can be handled.

The conditions on the north approach are almost exactly similar to those on the south side of the bridge, except that the length of the ocean wall is approximately

The information contained in this article was supplied by Sheet Piling, Inc., whose Larssen No. 2 and No. 3 sections will be used.

Immigration Statistics for Past Year

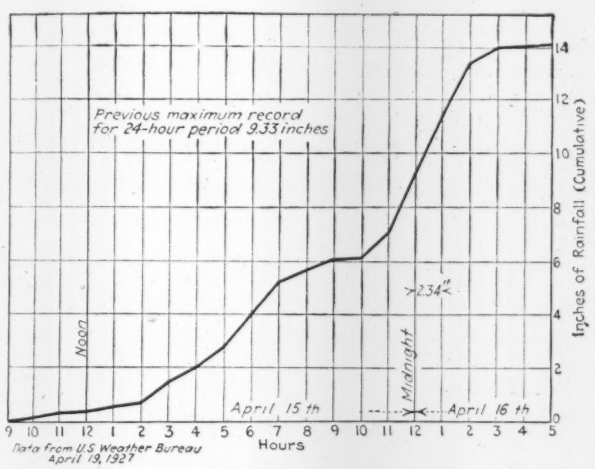
During the fiscal year ended June 30, 1926, according to a statement from the Department of Labor, a total of 496,106 aliens were admitted to and 227,755 departed from the United States. The net increase in population is 268,351, as against 232,945 in 1925. Of the aliens admitted 304,488 were newcomers, and 191,618 were non-immigrants returning from a short stay abroad or coming for a visit to this country. Of the aliens who departed 150,763 were non-emigrants. The remaining 76,992 were emigrants intending to make a permanent residence abroad. About three-fourth of present-day immigrant aliens are between the ages of 16 and 44. Common laborers predominate among the outgoing aliens. Three-fifth of the total emigrants leaving the country during the last fiscal year and reporting occupations were of this class; skilled workers ranked second among those having an occupational status, and servants were third. Of the aliens admitted 157,432 came in as immigrants charged to the quota; 150,299 as natives of non-quota countries, principally Canada and Mexico; and 83,754 as home-coming residents of America. The remainder came in temporarily for business or pleasure or else passed through the country in transit on their way elsewhere.

Notes on Heavy Rainfall April 15-16 at New Orleans and Vicinity

RAINFALL of 14 to 15 in. in two days, in the section surrounding New Orleans, La., combined with rising river floods and with unusually high tides blown in from the Gulf of Mexico by strong southerly winds, is reported to *Engineering News-Record* by Arthur M. Shaw, consulting engineer, New Orleans. Mr. Shaw very kindly sent the accompanying chart of the two days' rainfall as recorded by the local weather bureau and, under date of April 19, wrote as context:

Appreciating the interest which you have taken in the river situation at times of unusual high water, I have secured from the local Weather Bureau Office and other sources considerable information on the present floods, but having been advised by Dr. Cline's office that he had, at your request, sent you a long telegram covering the situation, I am forwarding only a graph which I have prepared of the rainfall at New Orleans on April 15 and 16, and some notes on other matters which Dr. Cline may not have covered in his telegram to you.

On the Scully reclamation project, 45 mi. S. W. of New Orleans, there is a standard rain gage carefully observed, which ran very nearly parallel with the official New Orleans



GRAPH SHOWING RAINFALL AT NEW ORLEANS, APRIL 15-16, 1927

Prepared from records of the New Orleans office of the U. S. Weather Bureau.

gage on which the enclosed graph was based. The reclamation project gage shows a total of 14.1 in. for practically the same hours as those covered by the New Orleans official gage, though the intensity of rainfall was considerably greater than in New Orleans from the hours of 4 to 8 p.m. of April 15, and somewhat less later in the night.

The rain gage maintained at Morgan City, which is 64 mi. W.S.W. and which is read at 7 a.m. each day, showed a total of 15.55 in. at the morning reading of the 16th. A reclamation project lying between the Scully project and New Orleans reported only 9 in. rainfall, but the records are not carefully taken at that point, and I am inclined to doubt their accuracy. The latter project was put under water by a combination of rain and wind tides which overtopped the levees. The Scully project was nearly all under water immediately after the rain, but the levees remained intact and the bulk of the area was drained before serious injury to crops occurred. This probably is one of a very few of the typical reclamation areas which was able to meet the situation with reasonable success.

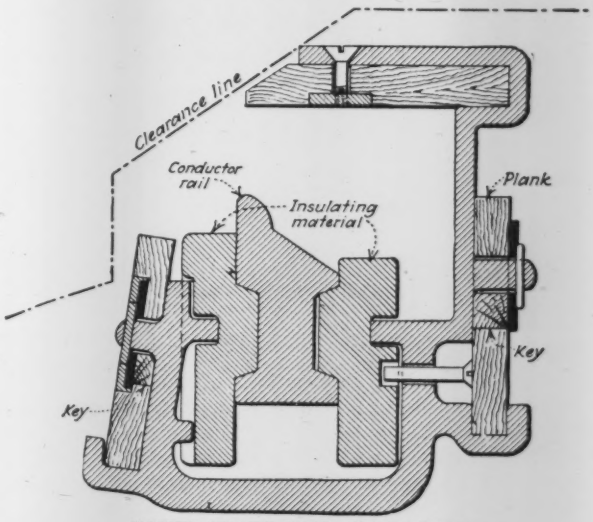
In addition to the unprecedented rain, the most of the reclamation lands suffered from an unusual high tide which has been blowing in from the Gulf ever since the first of the month. We have had no phenomenal wind storms, but the maximum velocity chart shows that the maximum wind velocity for any one day since April 2 has been 9 mi. per hr. or higher, and since April 2 the prevailing direction of the wind has been southerly. This has resulted in holding the water up on the prairie lands about 2.1 ft. above normal.

On the Mississippi Coast the high winds which accompanied the storm of the 15th and 16th resulted in a 4-ft. rise of tide along the coast. No damage was reported either to the sea wall construction or to the land development works of that section.

New Third Rail of Special Section for Paris-Orleans Ry.

WHILE the electrification of the Paris & Orleans Ry. from Paris to Vierzon is mainly on the overhead wire system, carrying 1,500 volts, it has also a 600-volt third-rail system on all four tracks to Juvisy and then on the two middle tracks as far as Bretigny, the end of the four-track district. This arrangement is due to the older installation of third rail and motor-car suburban service between Paris and Bretigny. All the main-line electric locomotives are equipped to take current either way.

A third-rail conductor of special section, as used on this line, is designed to give a better contact with the collector shoe and to minimize the troubles due to sleet and ice. From the accompanying drawing, it will be seen that the rail is of dumb-bell or double-head section, with a heavy upper head having a rib on which the collector shoe rides, while beyond this rib the head is inclined at a sharp angle. It is stated that with the ordinary contact of the two surfaces of shoe and rail there is continual arcing of current due to unavoidable irregularities of the surfaces. With the new rail there



NOVEL THIRD-RAIL CONDUCTOR

is practically a line contact, the multiple-panel shoe riding smoothly on the top of a narrow and rounded conductor.

The rail is 5½ in. high, weighs 84 lb. per yard and is in lengths of 60 ft. Its joints are welded, but expansion joints are provided at intervals of 8,200 ft., the rail being anchored midway between these joints. A splice plate extending up to the top of the rail carries the contact shoe and prevents it from dropping or catching in the expansion gap. Four copper bond cables connect the two rails across this gap.

From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

Concrete Slab of 19 Tons Lowered 18 In. by Melting Ice Cakes

By E. A. Nordstrom

Structural Engineer, Department of Public Works, St. Petersburg, Fla.

IN BUILDING the casino on the $1,000,000 municipal pier at St. Petersburg, Florida, a unique method was used in lowering a concrete bandstand platform. The second floor of the casino is a dance floor and as originally designed was to have no roof. The bandstand was built at one end of the dance floor and was approxi-

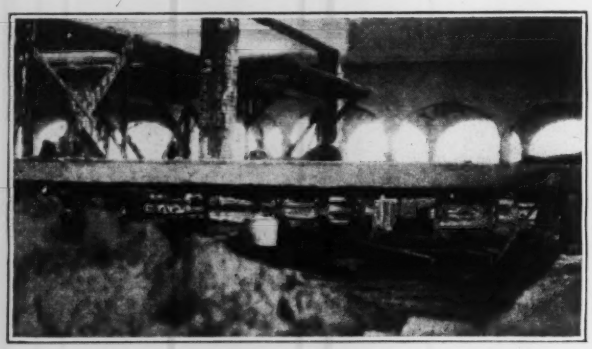


FIG. 1—BANDSTAND SLAB SUPPORTED ON CAKES OF ICE

mately 18 ft. x 20 ft. with an 8½-in platform of solid concrete with a floor tile and cement finish. The top of the platform was 34 in. above the floor.

It was subsequently decided to construct a roof over the dance floor to make it available for conventions, public gatherings, etc. When the roof was completed it was found that there was not sufficient room between the platform and the roof to install the sounding board at the back of the platform. It was, therefore, decided to lower the platform 18 in.

The platform was supported upon tile walls built around the outside edges. When these walls were broken through it was found that the formwork and supports were still in place. No record of the amount of steel used in the construction of the platform was available. When the platform was in its lowered position there would only be 7 in. clearance between the bottom of the slab and the floor. The platform weighed 19 tons. No jacks were available which could lower the slab with this small clearance and allow their removal. To break up the platform and reconstruct it in its new position was not only expensive, but would require too much time as the opening of the dance floor was only a week away. The scheme of lowering the platform with cakes of ice was then resorted to.

The platform was picked up on nine jacks and the outside walls were completely removed. As the bottom of the slab was rough and uneven new supporting piers were built of brick, their height being measured in each case from the tops of the original supports. Then, all the old supports were removed. The cakes of ice were then placed in position and the concrete slab lowered until its weight was picked up by the ice, wooden wedges being used to keep the slab level.

At the start of the work the slab was supported at nine points by stacks of ice symmetrically placed. The entire operation of lowering the slab took 36 hours. Because of the uneven melting of the ice and the length of time taken, the cakes of ice had to be replaced about 4 times. At the end of the operation only four cakes of ice were used, one placed at each corner of the slab. If it had been known at the start that there was sufficient steel in the slab for it to have carried its own weight without rupture when supported only at the four corners the time of lowering the slab could have been reduced to about 12 hours as the slab could have been lowered to the minimum height of the jacks using only four jacks. This knowledge, however, was only gained as the supports were removed and no apparent signs of rupture could be discovered. Readings were taken at regular intervals to check the level of the slab. The slab was securely braced on all four sides to prevent sliding, with just enough clearance to prevent binding.

It was anticipated that the ice cakes would not melt evenly. It was discovered, however, that the cakes of ice at the center of the slab were not melting nearly as fast as those on the outside. In order to prevent cracking of the slab these center stacks had to be burned out with a blow torch.

Some interesting facts developed regarding the way in which the ice melts under such conditions. For instance, when one cake of ice was stacked on top of another cake, instead of the two cakes freezing together and melting as one piece, they melted in the form of an arch as shown in *A*, Fig. 2. When wood wedges were used between the slab and the ice cakes the ice did not melt as fast under the wood as the rest of the ice cake did, leaving the wooden wedges up on a raised portion of ice or as shown in *B*, Fig. 2. The cakes of ice sup-

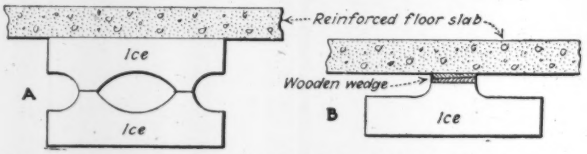


FIG. 2—DETAILS OF MANNER IN WHICH CAKES MELTED

ported the load without cracking when laid down lengthwise, while those cakes which were stood up on end failed by longitudinal shear and had to be replaced. This replacement was accomplished by temporarily taking the load on jacks until the new cakes of ice were in position.

Four laborers were kept busy with mops and pails to mop up the water. Just before the slab settled on the brick walls prepared for it, a layer of quick-setting grout was applied to the top of the walls which was squeezed out by the weight of the slab and provided an even bearing. The method proved to be successful. The cost was just one-fourth of what it would have been to break up the slab and rebuild it. The work, including repairs, was accomplished in three days, whereas, it

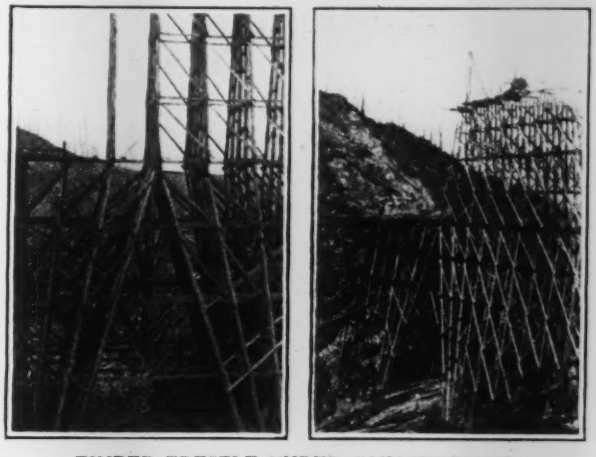
would have been at least three weeks before the platform could have been used if it had to be rebuilt.

The work was done by the engineering force of the Department of Public Works, of which Ernest Kitchen is the director.

Logging Timber Bridge in Northwest Is 204½ Ft. High

SOLVING a joint engineering and logging problem in the mountainous regions of the Pacific northwest has resulted in the recent completion by the Pacific States Lumber Co., Tacoma, Wash., of what is believed to be the highest timber bridge in the world. This bridge is across the Cedar River on the main line of the company's logging railroad between their operations in the mountains near Tacoma and their mill at Selleck.

Dense, old growth Douglas fir taken from the timberlands nearby is the construction material used in this bridge which is 204½ ft. high and 893 ft. long. It is double deck, the first deck being 124½ ft., and the second deck 80 ft. high. The width of the river bed is spanned by three leaning bents on each side of the stream which give it a clear span across the river of 80 ft. The highest bents which are 140 ft. long spread from a cap 14 ft. wide to a base 76 ft. wide; 508 140-ft. posts were used, scaling a total of 404,000 ft.b.m. These posts were from 12 to 14 in. thick at the tops and from 26 to 28 in. at the bottom. They were old growth Douglas fir, all more than 200 years old when cut. In addition to posts 432,000 ft. of sawed lumber was used in the bridge.



TIMBER TRESTLE UNDER CONSTRUCTION

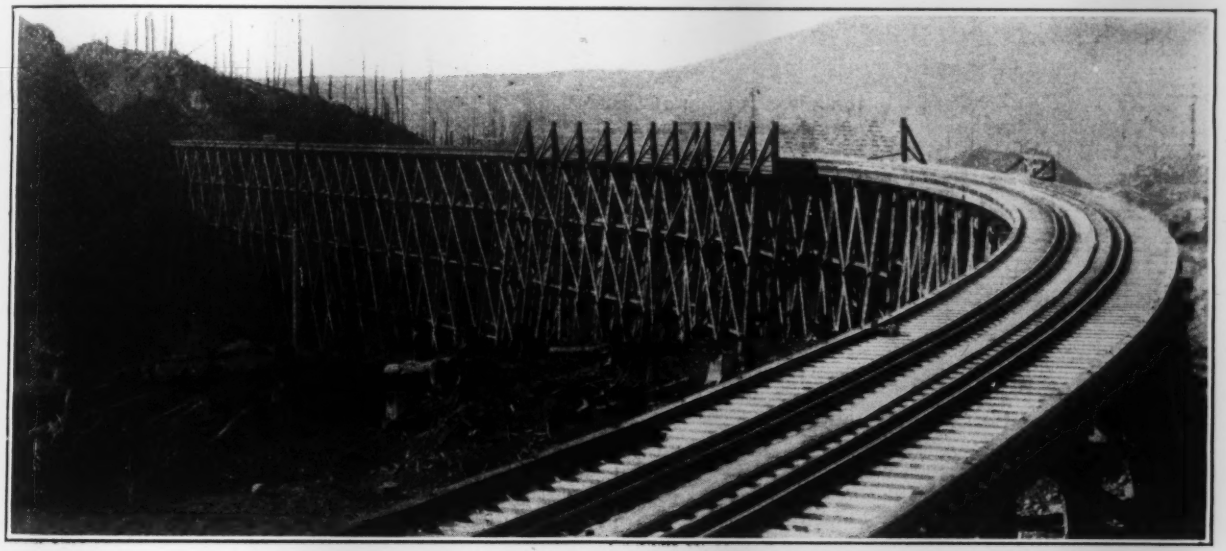
Left-hand view shows clear span above river. Leaning posts are 140 ft. long. Right-hand view shows erection of second deck, 80 ft. high. First deck is 124½ ft. high, at maximum.

All of the bridge posts were cut on the company's own operations. Logging is now going on eight miles away, 7,000,000 to 8,000,000 ft. being cut each month.

The bridge starts on a curve of 14 deg., but is on tangent in the highest section.

Practically the entire bridge rests on natural rock foundation. At the south end of the bridge it was necessary to make a cut through the rock, which is 65 ft. deep and 300 ft. long. To move this amount of rock and débris required 125 working days of 16 hr. each

The bridge itself was built in 110 working days of 8 hr. each. It was constructed by the Pacific States Lumber Company's own bridge crew under the direction of P. Cavanaugh and F. R. Van Campen, designers and engineers, and under the supervision of Walter Guske, woods superintendent. Material for this article was supplied by the West Coast Lumber Trade Extension Bureau of Seattle, Wash.



TRESTLE 893 FT. LONG COMPLETED AND IN USE

The timber barricade at the near approach supported on extra posts is to keep logs, which might roll off cars at the turn, from damaging a triple pipe water-supply line furnishing the city of Seattle with water from Cedar River.

Landing Locomotives at a Port Having 28-Ft. Tides

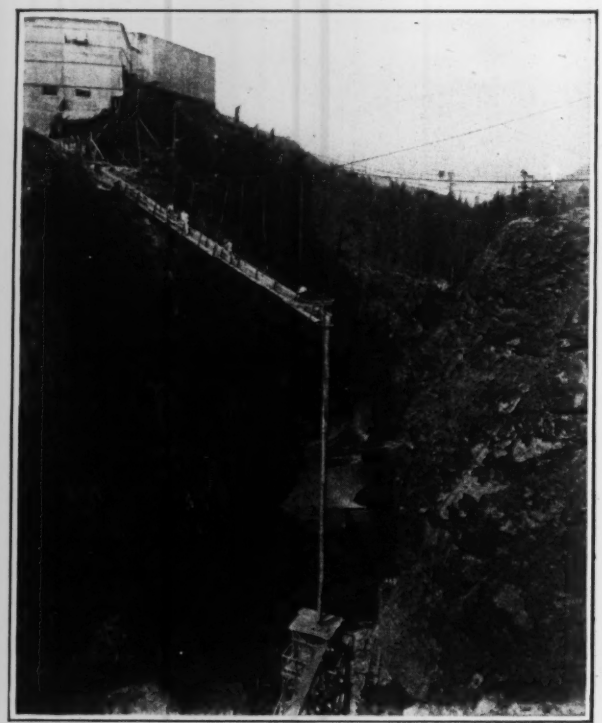
IN delivering two locomotives for the Argentine State Rys. at the port of San Antonio Oeste, 800 miles southwest from Buenos Aires, there was the combined difficulty of a 28-ft. rise and fall of tide in San Antonio Bay and a small crib pier with only two 5-ton locomotive cranes. There is 12 ft. of water at the pier during high tide, but practically no water at low tide. Various lighterage plans were considered and abandoned on ac-

count of the difficulties and cost and the lack of facilities for lifting heavy loads. Finally a small motorship was chartered, the "Scania," 182 ft. long, 14 ft. deep, with a net tonnage of 332 gross tons and a deadweight capacity of 600 tons. Its cargo booms were of only 2 ton capacity. With some difficulty the bulky cargo, knocked down for shipment, was stowed on board. A gin-pole 40 ft. long with heavy lifting tackle was then rigged and tested.

On arrival at San Antonio Oeste the gin-pole was erected on the deck, at the foremast, the tackle being operated by the ship's winches. As it was such a small ship and could not put a 26-ton load over the side without being likely to turn over, it was planned to put heavy timbers from the ship to the cars on the pier, landing the boilers on the ship's end of the timbers and sliding them down when the tide raised the ship. At the critical time, however, the ship failed to rise, but listed over to an angle of nearly 45 deg., so that only by prompt action was the ship prevented from turning over. However, with the assistance of the two pier cranes the unloading was safely accomplished in six days. This difficult job is described by Henriques Crawford in *Baldwin Locomotives*, published by the Baldwin Locomotive Works.

Concrete Makes 70-Ft. Vertical Drop at Glines Canyon Dam

IN DELIVERING concrete to the lower portion of the arch dam in Glines Canyon, Wash., concrete went from the mixer into a chute suspended from a cable across the canyon. The chute terminated in a vertical delivery



VERTICAL CONCRETE DELIVERY ON WESTERN DAM JOB

pipe, 15 in. in diameter, leading down to a steel-lined box that was 70 ft. below the chute at first and was raised as concreting progressed.

From the box, delivery to the forms was by means of another chute so that the concrete never dropped directly into place. In fact, the box itself was kept well filled with concrete during pouring so that a cushion was thus provided for receiving the batches as they came down the vertical pipe. When the forms reached the level of the chute, a trestle was built across the structure and the concrete was placed with buggies.

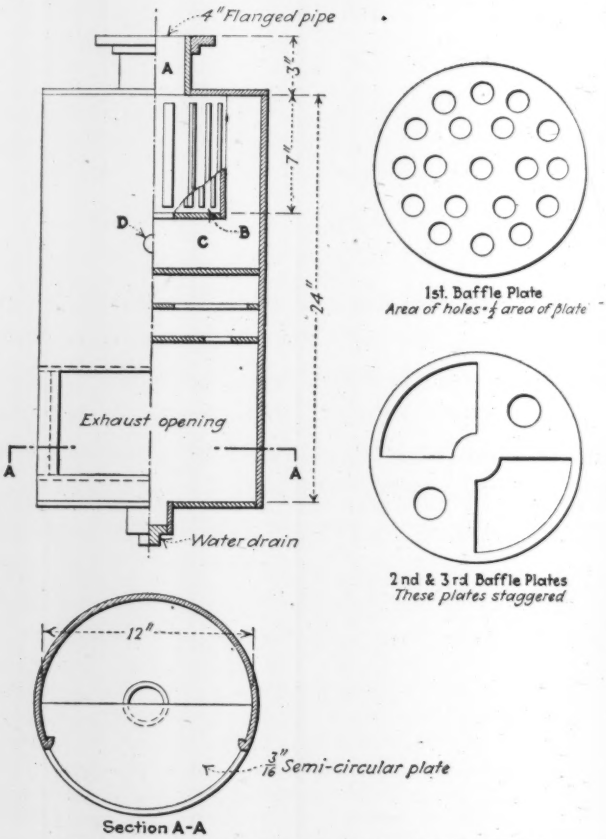
For the picture *Engineering News-Record* is indebted to W. B. McMillan, chief engineer for Thebo, Starr & Anderton, Inc., who designed and built the dam.

From Job and Office

Hints That Cut Cost and Time

Home-made Muffler Stops Exhaust Noise of Large Steam Hammer

IN DRIVING piles for the foundation of the Canadian Pacific Railway Company's pier B-C at Vancouver, B. C., it was necessary to work three shifts and as the pier is in the heart of the city the noise of the steam hammers was objectionable at night, and some hotels



DETAILS OF MUFFLER USED TO QUIET STEAM HAMMER EXHAUST

attempted to stop night driving by injunction. In order to decrease the noise of driving a muffler for each hammer was designed and built from material available on the job. When equipped with these mufflers the sound of the exhausts was so materially lessened that there was no further protest against night operation.

The accompanying sketch indicates the design and con-

From Job and Office

For Contractor and Engineer

struction of the muffler. Exhaust steam from the hammer enters through the flanged connection *A*, the first expansion taking place in the chamber marked *B*, where the steam is allowed to expand to approximately four times its own volume. This chamber consists of a short length of 8-in. pipe welded at one end to the cover plate and at the other end sealed with a circular, ⅛-in. steel plate. In the sides of this pipe are equally spaced, rectangular slots whose combined area equals the cross-sectional area of the pipe. Steam passes through these slots to the main expansion chamber *C*, of 12-in. pipe enclosed at the ends with circular steel plates. The flow of steam through this chamber to the exhaust opening is controlled by baffle plates which retard expansion without undue back-pressure.

Provision was also made for creating a vacuum by means of water injection. The muffler is in a vertical position when attached to the hammer exhaust, and thus water injected through the hole marked *D* falls to the sump at the bottom of the main expansion chamber whence it may be drained off. To prevent the water being driven out with the escaping steam semicircular plates were welded above and below the inside of the exhaust opening to deflect the water away from the outlet.

The muffler was designed by the engineering staff of the Sydney E. Junkins Co., Ltd., contractors for the pier; A. D. Wilder, vice-president and chief engineer.

Timber Bridge Rebuilt Under Traffic

By W. W. Zass
Construction Engineer, Arkansas Highway Department, Little Rock, Ark.

In replacing the superstructure of a timber bridge, 630 ft. in length with pile bents on 15-ft. centers the superintendent in charge displayed considerable ingenuity in employing a novel and effective construction plan which reduced traffic delays to a minimum. A

MOBILE RUNWAY KEEPS TRAFFIC MOVING DURING BRIDGE RECONSTRUCTION

single track runway approximately 30 ft. in length was mounted on skids and moved forward by a hand winch and cable as the work progressed. The caps, stringers and decking were replaced span by span, the runway allowing uninterrupted progress of the work. The only interruptions to traffic occurred when the work between bents was completed and the runway was moved forward to the next span. This moving operation was completed in each instance in about five to ten minutes time.

E. N. Sunderland of England, Arkansas, was the superintendent in charge.

Staggered Expansion Joints on Two-Strip Pavement

In Wichita Falls, Tex., where much two-strip concrete pavement has been laid in recent years, experience demonstrated that raveling occurred where the expansion joints crossed the center construction joint. The first remedy tried was to stagger expansion joints so that joints in each strip came opposite the slab centers of the other strip. The result of this practice was that a crack developed in the center of the slab opposite each

STAGGERED EXPANSION JOINT ELIMINATES RAVELING AND CRACKING ON TWO-LANE PAVEMENTS

expansion joint. In order to eliminate such cracks, the practice of staggering the expansion joints only 2 ft. and in some cases only 12 in. as shown in the foreground of the accompanying illustration was started. No dowels were used in the length between expansion joints although they were used at all other points along the slab. A piece of roofing paper was inserted in this length between joints to break any possible bond. Preformed expansion joints were used in all cases.

Some 3,000 to 4,000 yd. of pavement have been laid with such an expansion joint arrangement without resulting in the raveling and cracking that had occurred previously. F. M. Rugeley is city engineer of Wichita Falls. The pavement shown in the illustration was placed by L. E. Whitham & Co., paving contractors, Wichita Falls, Tex.

News of the Week

Current Events in the Civil Engineering and Contracting Fields

Mississippi Record Flood Crest Passes Memphis

Highest Water in History of River—Levee Breaks Flood Large Areas —Danger Ahead for Several Weeks

THE highest flood on record is passing down the Mississippi River. The situation changes from hour to hour so that reports written as this one some days before it reaches the reader are subject to correction. There can be reported here only the instant condition, but this report, as of April 27, should make it clear that the flood is following a familiar history of submergence. No attempt is made to record losses or areas of the flood; any figures as to these features at the present time are obviously the rankest of guessing.

The general condition is that the crest of the flood, which made itself manifest about April 1, is passing Memphis and is moving south at gages higher than ever recorded. Levee breaks in the main river and in the tributaries have caused backwater flooding of enormous areas, but they have at the same time relieved local conditions, as, for instance, at Arkansas City on the west bank of the river, where the crest fell 1½ ft. the day after the break in the levee at Mounds Landing directly opposite. But the peculiarity of the Mississippi basin is that all of the water which overflows or breaks through the levees eventually reaches the river lower down. The major river valley of the Mississippi has been built up above the neighboring country so that all drainage is away from the river to major streams which, roughly paralleling the main river, enter it farther down. When a break in the levee occurs, therefore, the paralleling river valleys are filled with backwater, sometimes reaching up to the back of the existing Mississippi River levees and that water slowly passes down the secondary valley to join the main river later. This passage may take so long that it reaches the river after main crest has fallen, but, again, it may reach the river when a secondary crest is arriving and thereby raise the crest. Conditions below the breaks, therefore, can only be guessed at; upon synchronism of these returns to the main river, upon rainfall, upon condition of the discharge of the lower tributaries will depend the height to be reached in the lower river. However, all authoritative predictions look toward a gage at New Orleans higher than ever before reached.

To date, the major levees of the river have broken in four places—at Dorena, Walnut Bend, Laconia, all on the west bank, and at Stopps Landing on the east bank above Greenville. These are located on the accompanying map. Only one of these, the Stopps Landing break, occurred when the rear of the levee was dry. On the west side, back water had reached the levee in each case. It happened, too, that the one on the east bank, at Stopps Landing, is the most serious of the four, indeed one of the most serious that has ever occurred on the Mississippi, because it has flooded the city of Greenville, a town of 12,000 inhabitants.

History of the Flood—It is difficult at this date to analyze the causes of the flood. Certain facts, however, appear from a study of the gage heights and of the rainfall records. The present Mississippi River flood, as all such floods are, is a combination of synchronized events. A heavily flooded Ohio was met lower down by flood flows in the Arkansas and the White, due to rain coming at just the right time in the corner of Oklahoma, Missouri and Arkansas to bring the rivers draining that territory in concurrent flood with the Mississippi at their mouths. To add to the difficulties the lower Mississippi was itself heavily loaded by the extraordinary rainfall in the area centering on New Orleans when on Apr. 15 the total fall in 24 hr. was 14 to 15 in.

These general conditions can be followed by a study of the gage table given herewith. It will be seen that the Ohio has been high all winter. The upper Mississippi and the Missouri have been normal, in fact lower than normal most of the winter, so they contributed but little to the present flood. The Ohio floods, however, seem to have been the major cause of high water in the Mississippi since last fall.

The Mississippi at Memphis, for instance, had a maximum stage of 25.1 last September, the highest of record in Memphis in 54 years with one exception. In October it was the highest since 1871. In November it was the highest, with two exceptions, since 1871, and in December the highest since 1871 with two exceptions. On Jan. 12 the Mississippi at Memphis had a gage 37.7 where 35 is flood height. It dropped off in the next 10 days to 20.9

Asheville A.S.C.E. Meeting Well Attended

Nearly Five Hundred Enjoy Spring Meeting—Student Chapters Emphasized

(Engineering News-Record Staff Report)

Almost five nundred members and guests registered at the spring meeting of the American Society of Civil Engineers held at the Kenilworth Inn, Asheville, N. C., April 20 to 22. It has been some years since the society has had a resort hotel where practically all of the attendance could be housed under one roof and where there was little opportunity for dispersal to attend to private business and social engagements. The return to the earlier practice of having at least one such meeting a year, if an experiment, proved worth while for the large number registered were together practically all of the time, the opportunities for social contact were great and the attendance at the technical sessions more than satisfactory.

No business was transacted at the meeting. There were two general sessions, one devoted to sanitary matters, the other to highways, and the sanitary, construction and power divisions held simultaneous sessions one morning. The rest of the time was devoted to excursions to nearby power developments and the new suburban resort community being built at Lake Lure, near Asheville.

Session with Students

One afternoon was given over to the activities of the student chapters. Over a hundred technical school undergraduates from a dozen or more of the institutions in the South were present throughout the meeting. This feature had been developed by Director Braune, of the University of North Carolina, who is chairman of the Board's committee on student chapters. These young men, all members of the various student chapters, attended all of the

(Continued on p. 708)

Engineering Fifty Years Ago

From *Engineering News*, April 28, 1877

Annual Convention of Am. Soc. C. E. at New Orleans April 23, 1877

PROF. FORSHEY spoke of Humphrey's and Abbott's formula in regard to the flow of water in the Mississippi, and of the necessity of further investigation into the flow of water in large rivers. Mr. Corthell had not prepared a formal paper, but offered many interesting remarks on the works at South Pass, as being on the general problem of river hydraulics. Mr. Corthell was exceedingly frank in his remarks, confessing that in a measure they had been over-confident but they were learning every day. The problem was by no means solved. Many things that they had tried had failed without apparent reason.

on Jan. 22, but rose again to 37.8 on Feb. 12. It dropped off again March 2 to 30.3, but from then to March 30 there was a steady rise, reaching 41.4 on that date. It looked as though the crest of the flood had been reached at that time because it dropped off to 39.8 on Apr. 5, when the present record rise started and has continued steadily since. The Mississippi, therefore, never really went down from September on. Any of the tributary water reaching the Mississippi at that time was reaching nearly a bank-full river.

The western tributaries of the lower river were low all last fall, but their watersheds were subjected to a number of rains in January and February which caused them to rise, at times almost reaching bank height. Thus on Feb. 28 the White River at Clarendon was at stage 26.1 or 4 ft. below bank, though the Arkansas at Little Rock was only at stage 4.0 or 19 ft. below bank. During March the White stayed high and the Arkansas began a continued rise with one drop-off the first part of April. At this time, that is late in March and early in April, torrential rains in southeast Kansas and northeast Oklahoma started to flood both streams and they continued to rise steadily, the Arkansas at Little Rock reaching on Apr. 21 stage 32.6, which is a record since 1844, though there is record of 2 ft. higher in the famous 1833 flood. The following figures will show how much water was coming down the Arkansas:

Date	Fort Smith	Little Rock
1833	38.0	34.6
1844	34.3	32.6
1922	27.5	23.2
Apr. 16, 1927	36.7	Apr. 21, 32.6

It will be noted that the Arkansas River high water this year is greater than in 1922, but that in that year, the previous record flood in the Mississippi, the flood heights below the Arkansas and the White were made without any great contribution from those two rivers.

The main gage table herewith shows the conditions in the Red River by the gages at Shreveport. It will be noted that while that river was fairly high, it did not commence to rise until about the middle of April, so that the crest of its high water has not yet reached the Mississippi. It will, therefore, continue to contribute to the flood heights in the lower river.

Levee Breaks

Generally speaking, the river is proceeding to the Gulf between the lines of levees established by the federal government under the control of the Mississippi River Commission and by the various local governmental authorities along the back waters of the tributary streams. The reader interested in a succinct statement of the Mississippi levee system is referred to an article by Col. Charles L. Potter, Corps of Engineers, U. S. A., President of the Mississippi River Commission, in *Engineering News-Record*, March 26, 1925, p. 508. As stated there, the commission's flood control policy is to confine between levees the flood flow as nearly as possible to the bank-to-bank width of the river. The commission has a system of levees according to this intention from Commerce, Mo., to below New Orleans, with numerous extensions of the levee up the tributary streams. These extensions were not originally in the control of the commission, but have gradually been taken over. There exist, however, many miles of other levee not under governmental control, either as backwater levees protecting certain districts and towns from overflow from the rear or as tributary levees extending up smaller streams.

The commission levee is designed to a grade 3 ft. higher than the highest flood, with a section to take care of pressures so imposed. However, this height has not been reached in all sections of the river. Work is continually going on. According to the report of the commission in 1925, for instance, 88.7 per cent of the estimated final content of the government levees as then planned had been completed. Most of the secondary levees are not of the same height and section as the government levees. In addition, of course, the river is protected by low water revetments, dikes, etc., but in methods used then in fighting a flood and these methods are quite the same today. As noted earlier in this report, there have been but four breaks in the main levees that have been reported up to the time this was written. In addition there is report of a break in one of the levees below New Orleans, details of which are not available.

There was some concern at Cairo for a while, but apparently the danger there is now past. The first break was at a steamboat landing called Dorena, on the west bank, some 30 miles south of Cairo. Here contractors were rebuilding the levees and there was in front of the machine a low place to be brought up to grade and section. Protection by sacks and plank was attempted, but the crest became too high, the levee was overtopped and failed. This augmented the backwater which had developed by the bypassing of the St. John Bayou, as shown in the accompanying map. At New Madrid, this backwater reached the local levee protecting the town, and this levee failed. The backwater proceeded to flow down into the St. Francis basin. At Walnut Bend, on the west side of the river near the mouth of the St. Francis, a con-

GAGE HEIGHTS AT MISSISSIPPI VALLEY POINTS

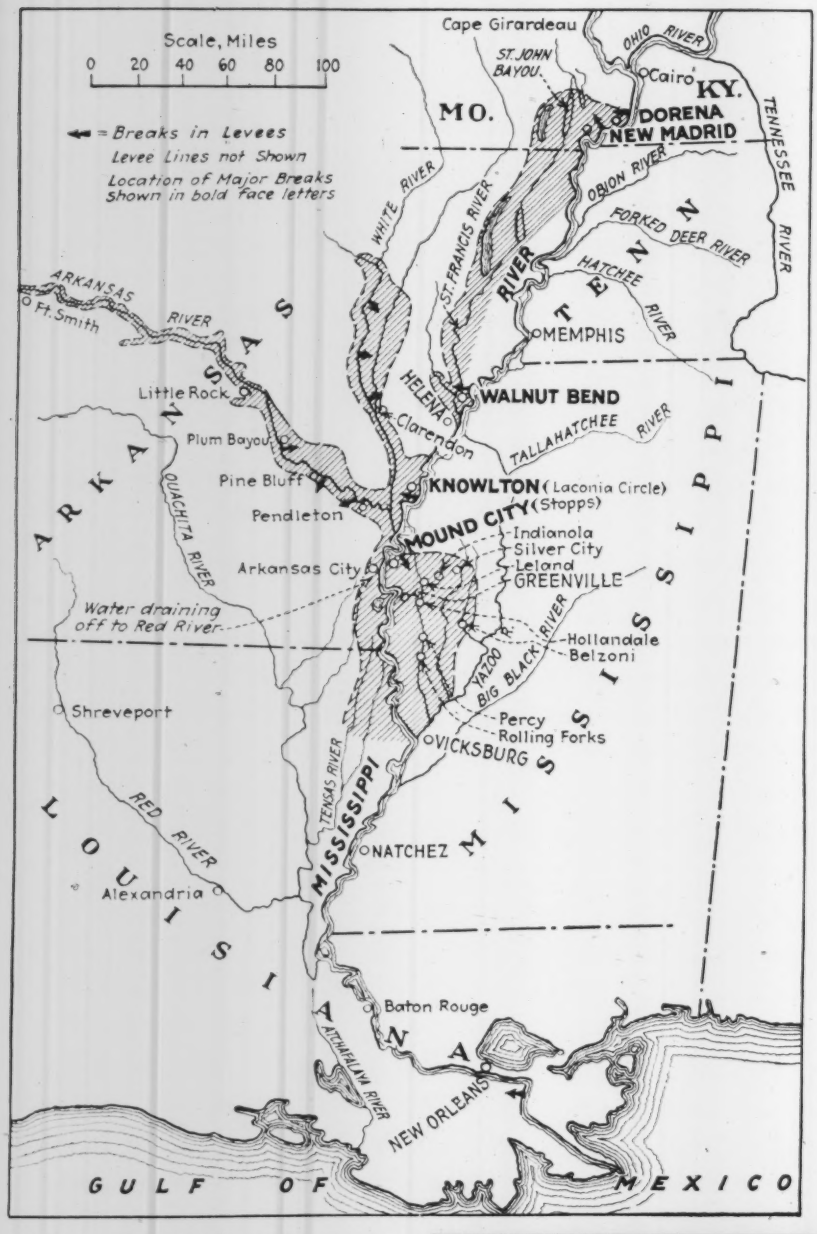
	Upper Mississippi River, Davenport, Iowa	Ohio River, Evansville	Missouri River, Kansas City	Mississippi River, St. Louis	Mississippi River, Cairo	Mississippi River, Memphis	White River, Clarendon	Arkansas River, Little Rock	Mississippi River, Vicksburg	Red River, Shreveport	Mississippi River, New Orleans
Flood stage†	15.0	35.0	22.0	30.0	45.0	35.0	30.0	23.0	45.0	29.0	17.0
1913	....	....	....	....	54.7	46.5	30.4	6.9	52.2	12.3	19.3
1922	....	....	....	....	53.6	42.3	30.8	23.2	55.0	31.0	22.6
December	....	35.8	3.5	....	....	....	27.4	....	....	....	....
January	....	....	8.2	4.0	....	....	27.5	....	....	....	....
January 1	....	....	....	....	....	....	....	9.2	....	....	....
January 12	....	....	....	....	....	37.7	....	....	....	....	....
January 22	....	....	....	....	....	20.9	....	....	....	....	....
January 26	....	....	....	....	....	....	....	9.2	....	....	....
January 31	....	....	....	....	....	....	....	17.8	....	....	....
February	....	44.5	10.0	....	....	37.8	31.1	4.0	....	....	....
March	....	....	....	27.3*	....	....	....	....	....	....	....
	....	36.0	11.8	20.3**	....	30.3	28.6	17.5	....	....	....
April 1	12.2	38.5	12.5	23.1	49.4	41.0	28.9	7.5	49.6	19.2	18.3
April 2	12.2	37.3	14.8	27.7	49.9	40.6	28.9	7.5	50.1	18.8	18.4
April 3	....	....	....	....	....	....	....	....	....	....	....
April 4	11.6	35.9	13.7	30.0	51.6	40.0	28.8	10.8	50.6	18.1	18.7
April 5	11.3	35.4	13.2	31.0	52.3	39.8	28.7	15.9	51.0	18.4	18.7
April 6	11.0	35.0	12.8	30.7	52.8	39.9	28.6	17.5	51.2	18.0	18.9
April 7	10.7	34.6	12.6	30.0	53.1	40.2	28.5	17.4	51.5	18.5	19.0
April 8	10.4	34.5	12.8	29.1	53.2	40.5	28.5	16.4	51.7	19.1	19.1
April 9	10.2	34.4	13.1	28.5	53.0	41.0	28.6	15.5	51.9	19.9	19.2
April 10	....	....	....	....	....	....	....	....	....	....	....
April 11	9.6	33.7	14.4	28.8	52.3	41.9	28.6	18.8	52.3	21.6	19.4
April 12	9.4	33.5	17.2	29.6	52.3	42.3	28.7	19.0	52.4	22.4	19.5
April 13	9.2	34.3	17.2	30.7	52.7	42.5	28.8	19.4	52.5	23.3	19.6
April 14	9.0	35.3	16.5	31.9	53.6	42.6	29.9	21.4	52.7	24.9	19.7
April 15	9.0	36.0	17.1	33.3	54.6	42.7	30.1	24.6	53.0	26.5	19.8
April 16	9.0	36.3	20.0	34.0	55.7	43.5	30.7	26.9	53.6	27.3	20.2
April 17	....	....	....	....	....	....	....	....	....	....	....
April 18	9.0	35.6	20.4	34.4	55.3	44.3	36.6	30.4	53.9	28.7	20.1
April 19	9.1	34.7	22.0	34.6	55.3	44.4	43.6	31.0	54.2	29.3	20.2
April 20	9.3	33.7	24.0	34.8	56.4	44.7	....	31.7	54.5	30.1	20.3
April 21	9.6	33.0	24.8	34.9	56.2	45.3	....	32.6	54.9	....	20.5
April 22	....	....	....	....	....	....	....	....	55.4	....	....
April 23	9.5	30.6	20.6	35.2	55.3	45.9	....	31.8	55.2	32.8	20.7
April 24	....	....	19.0	35.6	54.8	45.9	....	31.2	54.8	34.1	20.8
April 25	....	....	....	35.8	....	....	....	....	....	....	21.0
April 26	....	....	....	36.1	....	....	....	....	....	....	20.9

* March 22. ** March 31.

† Flood stage is an arbitrary level, approximately bank-full condition.

time of flood it is only the levee which is of importance.

This whole system of levees has been subjected to the pressure of the flood and under constant surveillance by guards and in most cases under constant defense by the familiar means of sandbagging, earth fill, etc. The article by W. W. DeBerard, Western Editor of *Engineering News-Record*, in *Engineering News-Record*, May 18, 1922, p. 814, describes thoroughly the tractor was putting in a new levee. He had cut through the main levee and put a loop around behind to protect the country while the new levee was under construction, but the pressure became too great for it, broke through the newly placed loop, and extended the backwater farther south. At Knowlton, in the White River district, just north of the mouth of the White, there was a cave in the so-called Laconia circle. There had been a program for



ABOVE: THE MISSISSIPPI VALLEY BELOW CAIRO ON APR. 27

a raised levee at this point, but the appropriations had not been met by the local bodies and the levee was below grade and section. This completes the list of breaks on the west side of the Mississippi system, though, as indicated on the map, there were numerous breaks in the minor levees on the Arkansas and White which permitted large areas to be flooded there. The flow from these valleys is passing south behind the west bank levees toward the Red River valley.

E. U. Stevenson, field engineer of the Portland Cement Association, reports the following condition in the Arkansas River valley as of April 22:

"With the flood at its crest on April 20 and with a rainfall for the week of 8.9 in. hope of saving the pumping plant of the Little Rock water supply from inundation was almost abandoned. For five days from 600 to 1,000 men had been working constantly to maintain a sand-bag dike surrounding the plant. Laborers sent by local contractors together with state convicts stuck to the job 9 ft. below the elevation of the water outside the dike and turned defeat to victory. Many other towns suffered curtailment of water supply. Van Buren's wells and filter beds were flooded on April 21 and emergency pumps were later installed and water was chemically treated. Fort Smith's water plant closed on April 15 due to floods. Early in the morning of April 21 a steel truss bridge at Little Rock, leased to the Missouri Pacific Ry., failed. The south span and draw span went into the river. The piers are of steel cylinder type. Whether failure was by overturning or undermining cannot now be determined. The bridge was built by a British company in 1873. The superstructure had been strengthened several times to care for increased loads.

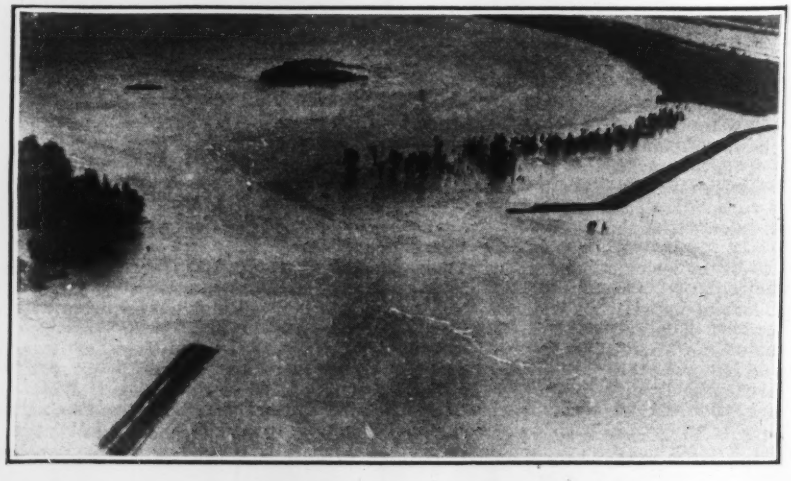
"The railroads over the state exerted every effort to help flood sufferers. There are probably 3,000 box cars now in service as refugee homes.

"Perhaps the most outstanding tragedy in a week of tragedies was at Clarendon where the failure of White River levee in the early morning of April 20 flooded the town in 10 min. to an average depth of 10 ft. Property losses are enormous in the flooded sections of Pine Bluffs and north Little Rock, where blocks of business and residence sections are covered with water.

It was reported on April 27 that Arkansas City had been reached by the backwater and was under water.

Greenville Inundation

The most serious break in the whole river occurred at Stopps or Mound Landing on the east bank of the river, 20 miles north of Greenville, Miss., and just opposite Arkansas City., Ark. This is



BELOW: LOOKING DOWN FROM AN AIRPLANE ON THE STOPPS LANDING LEVEE BREAK WHICH FLOODED GREENVILLE

a main line levee of the Mississippi River Commission. Concerning it Col. Potter, President of the Commission, wires: "Break at Mounds only one ever had in full commission grade. Was caused by eddy below spur-dike intended to protect angle in levee. Spur melted so suddenly that main levee was attacked before any remedy could be applied." What happened is best reported by a telegram from E. J. Tucker, of Vicksburg, as follows:

"At 8 a.m., Apr. 21, the Mississippi River broke the levee at Mound Landing on the East or Mississippi side at a point opposite Arkansas City, Ark. Since Apr. 1 the third Mississippi district has been on special high water service, patrolling the levees with as perfect an organization as is conceivable and all levees in the district have been subjected to aerial inspection twice daily by Maj. John C. H. Lee, engineer in charge. The section at which the break occurred was recognized as a weak point and for more than a week every available man had been sacking and strengthening the section. They were working at the moment of the break and a number of men went in with the levee and were killed. The break came with the river at a stage of 60.5 ft. on the Arkansas City gage, [exactly the Commission provisional grade, according to a wire from Major Lee, U. S. District Engineer at Vicksburg] after a night of heavy rain and high wind and following an unprecedented 24-hr. rise of 1.1 ft. The previous record was 58.0 ft., at the crest of the high water of 1922. The break came at 8 a.m. By 9:30 it had widened to ¼ mile and at noon it was reported to be ½ mile wide. At the time of the break water was pouring over the top of a 6-mile stretch of levee 4 miles farther upstream. This 6-mile stretch still stands apparently undamaged except for wash on the land side.

"In Greenville, 18 miles overland below Mound Landing, an effort was made to raise the municipal levee and to keep out the crevasse water. At 3 a.m., Apr. 22, water was coming over its entire length 1 ft. deep and work was abandoned. The last train out of Greenville left at 5 p.m. Greenville is a city of 12,000 inhabitants and all are being brought to Vicksburg by boat. In addition to Greenville, the following cities will be affected: Leland (already under 3 to 5 ft. of water), Hollendale, Morehead, Indianola, Belzoni, Percy, Silver City, Rolling Fork, Meyersville, and a number of smaller towns."

A telegram received from W. E. Elam, of Greenville, Miss., dated Apr. 25, 6:21 a.m., says, in part:

"The scene of a present-day break is awe inspiring, but the back country is hell. To a people accustomed to breaks and knowing from its locality just where the water goes, no great alarm is felt. In this instance no one knew what effect the network of drainage ditch spoil banks would have on the crevasse water and thought little of it until the water began to show up. It was then seen that prompt measures would have to be taken to save life. Further to complicate matters a desperate topping fight was being waged near Greenville by an organization that knew everything they had was going to be inundated by the water from the break. To stop and permit a break to occur here would cause untold loss of life. Rapid changes in organizations were made to fill the places of those whose families were in the most danger. Men and material were rushed to the scene until the water stopped going over the main levee. Then when the water showed a definite fall, men and material were transferred from the main levee to the protection levee on the upper side of Greenville.

LOOKING DOWN ON FLOODED GREENVILLE FROM AIRPLANE

"The break water came against the protection levee about 11 o'clock that night and rose rapidly until the water ran over the topping. The rate of rise at first was about 1 ft. every 15 min. but it slowed down to about 3 or 4 in. in every 15 min. when the work failed. The city of Greenville is practically all under water from a few inches to 8 to 10 ft. The Mississippi Power & Light Co. is maintaining service for lighting, but the water and sewer plants are out of commission."

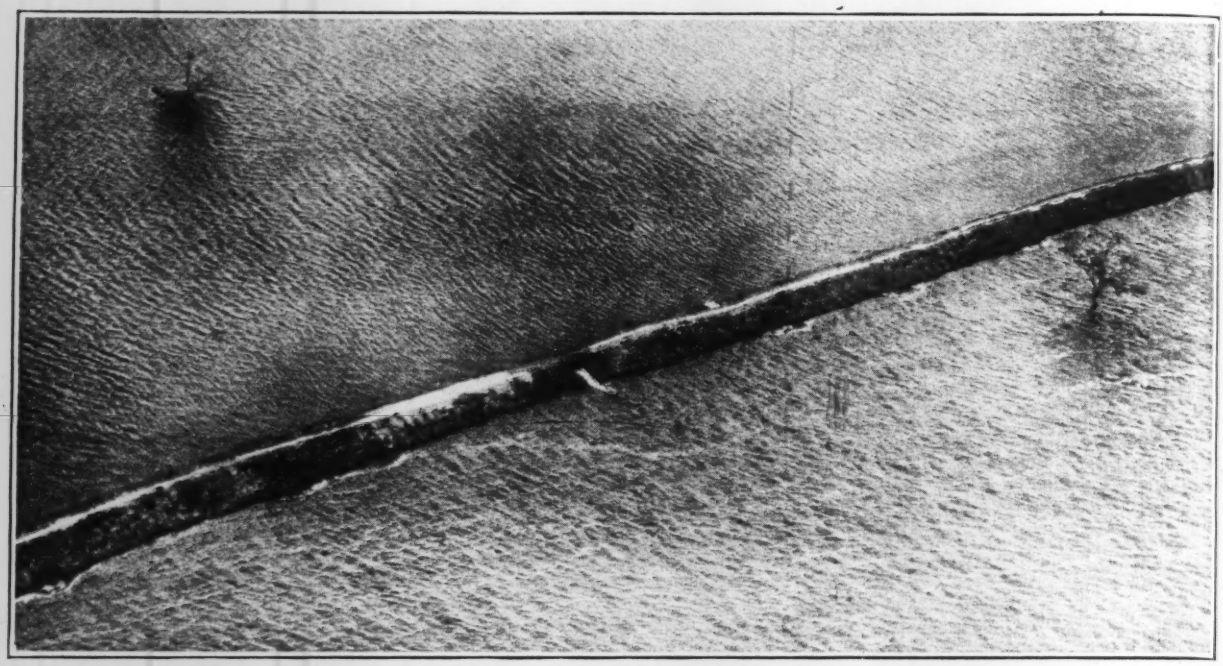
The Greenville flood water will have to pass down the Yazoo valley and out of the Yazoo at its mouth at Vicksburg. Just when it will reach Vicksburg is not known. At Vicksburg proper, Mr. Tucker reports: "The wholesale district of Vicksburg, the only part of the city subjected to overflow, is protected by a reinforced-concrete sea wall. At the time of construction, in 1924, it was announced that there were sewers under the wall which could not be located and stopped, because some of them had been laid before the Civil War and long since been abandoned and forgotten. This, the first high water since the sea wall was built, is blowing out old sewers and causing some concern. So far no serious damage has been done. Return water is expected to produce a stage of 58.0 ft. at Vicksburg and a corresponding stage (3 ft. higher than ever before) at points below, provided the levees hold. Most of Vicksburg is on high ground."

Prof. J. H. Bateman, of Louisiana State University at Baton Rouge, reports the following condition at that city:

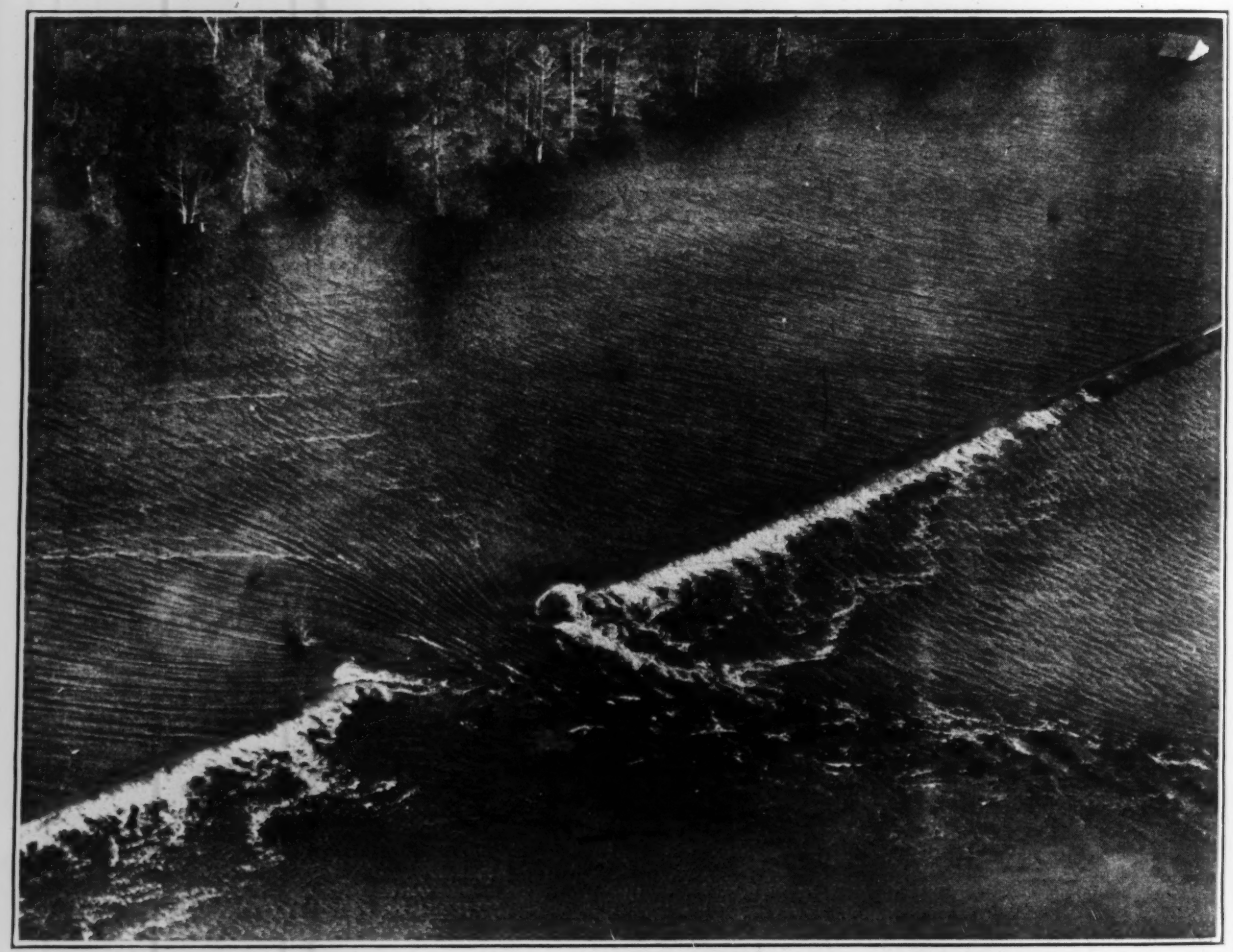
"All Louisiana levees are still holding although high winds the night of April 21 caused considerable caving by wash. Serious caving occurred April 21 north of Donaldsonville and large numbers of men were able to repair the damage by new filling, sand bags and timber revetment. A similar situation prevailed at Geismar but was handled by sand bags and timber revetment. Water is rising rapidly at Baton Rouge. The river front has been strengthened by gravel embankments, sand bags and timber revetment. The Yazoo & Mississippi Valley R.R. has raised long stretches of its tracks to prevent interference with train operations. The passenger tracks at the local depot platform are now lower than the water level, but are believed to be fully protected. The levee immediately below Baton Rouge was subjected to severe wave wash but has been strengthened by revetment and is now safe. Levees on the west side of the river near Baton Rouge are holding and long stretches are being sand-bagged to care for addtional rises."

On April 26 the Governor of Louisiana, acting under advice of an engineering board and with the approval of the Mississippi River Commission, recommended the cutting of the levee at Poydras on the east bank just below New Orleans in order to lower the flood height at New Orleans. This is the location of a crevasse in the 1922 flood. It is expected that this artificial spillway will be made before the end of the week.

This is the first time that an official order has been signed to open the levee. Poydras is also the site of the proposed permanent spillway New Orleans has been long advocating.



BEGINNING OF LEVEE BREAK (ABOUT MIDPOINT OF PHOTO) ON THE WEST BANK LEVEE FLOODED BY BACKWATER



AIRPLANE VIEW OF ARKANSAS LEVEE TAKEN SHORTLY AFTER CONDITION SHOWN ABOVE WAS PICTURED

Greatest damage to state highway roads and bridges among the Mississippi Valley states was sustained by Missouri. According to T. H. Cutler, chief engineer of the State Highway Department, the state has lost or had damaged six highway bridges of some size. A bridge over the White River, 12 miles from Springs, which consisted of two 60-ft. spans and two 180-ft. spans supported on concrete piers and concrete and bents and carrying a wooden floor with approach fill, suffered the loss of one 60-ft. span and one 180-ft. span.

Another bridge over the White River at Branson consisting of two 190-ft. spans and a 250-ft. approach trestle on the north side and a short one on the south with concrete floors on the main spans escaped other loss than that of the trestle on the north side. Other damage to bridges included two county road bridges over the White River which were lost and the loss of a 100-ft. steel bridge over Turkey Creek in Hollister.

In Taney County, the bridge at the edge of Forsythe, consisting of three 160-ft. spans and two 60-ft. spans, sustained considerable damage which cannot yet be accurately determined. About 10,000 cu.yd. of approach fill have been washed out, the south abutment wrecked, and the 60-ft. span which forms the south approach has settled about 4 ft. on the upstream side and 2½ ft. on the downstream side.

The north approach fill to the Waverly bridge across the Missouri River was damaged for a stretch of 1½ miles due to the flood waters coming through a broken levee and running over the top of the slab. The fill was made from sandy soil taken from nearby borrow pits and did not withstand the scouring action of the water.

The Mississippi State Highway Department reports 300 miles of main road constructed either of gravel or 18-ft. concrete completely submerged with heavy currents flowing across some of it.

O. L. Hemphill, state highway engineer of Arkansas, reports damages to roads and bridges all the way from several hundred thousand to more than a million dollars. Most of the damage is to small structures and embankments, with but one major structure reported out so far. This is an old bridge over the Ouachita River, west of Hot Springs. About 700 ft. of embankment on the Little Rock to Hot Springs concrete road was washed away with heavy damage to other embankments on this road.

In the extreme western part of Kentucky several miles of state highways are under water. No bridges built to modern standards have failed and only the lighter structures built by counties have been destroyed.

The principal damage in Oklahoma to state roads is through excessive washing of gravel roads. Loss of washing of gravel roads.

Loss of principal structures by railways has been negligible throughout the area. The Missouri Pacific reports several divisions under water and in many places dumps and tracks washed out. The only major structure lost to date is two spans of the bridge across the Arkansas River at Little Rock, which failed April 21. However, the Missouri Pacific has another bridge across the river at this point which is safe, so traffic continues.

A further wire, dated April 26 is to the effect that the Missouri Pacific lines are all open to traffic except a few local points covered by embargoes.

The assistant chief engineer of the Chicago, Rock Island & Pacific, wires as follows:

"Flooded conditions have not affected the Rock Island except in Arkansas on the main line between Brinkley and Devalls Bluff and between Little Rock and Houston on the Hot Springs line. Between Little Rock and Haskell high water submerged the track and washed out telegraph lines and damaged station buildings.

A. F. Blaess, chief engineer of the Illinois Central wires:

"The break in the Mississippi River levee fifteen miles above Greenville has flooded an area of the Mississippi Delta bounded on the north by Rosedale and on the south by Vicksburg and extending twenty to thirty miles eastward from the river. We have around two hundred miles of line in this area now under water. While this includes some main line of the Yazoo & Mississippi Valley, the main line of the Illinois Central proper is not affected."

John E. Ericson

City Engineer of Chicago, Who Died April 16, Held Position Nearly Thirty Years

John Ernst Ericson, city engineer of Chicago for many years and identified closely with the city's engineering developments, whose death on April 16 was noted in these columns April 21, was born in Sweden, Oct. 21, 1858. After graduating from the Royal Polytechnic Institute of Stockholm in 1880 he came to the United States, where his first employment was in 1881-1882 as resident engineer of construction on the Toledo, Cincinnati & St. Louis Ry. (now the Clover Leaf division of the New York, Chicago & St. Louis Ry.). In 1882, also, he was engaged on survey work under the U. S. Corps of Engineers. In 1884, he became a draftsman in the Chicago water department and in 1885 was made assistant engineer of the bureau of engineering. He was assistant chief engineer of water-works at Seattle, Wash., 1889-90, and then an assistant engineer of the Chicago Sanitary District in 1891-92.

Returning to city service in Chicago as assistant engineer in the Bureau of Engineering in 1892, he was appointed first assistant city engineer in 1893 and city engineer in 1897. This last position he retained, with a few interruptions, until his death. During 1903-05 he was on leave, to serve as a member of the Board of Local Improvements. Under Mayor Thompson he resigned in October, 1919, but acted as consulting engineer to the commissioner of public works until 1923, when Mayor Dever reappointed him as city engineer.

His name is associated specially with the great developments in the water supply system of Chicago. One of his first steps was the connection of the water-works of several annexed towns and villages so as to form a unified system. New lake and land tunnels and a dozen large pumping stations were built under his direction, besides vast extensions and improvements in the distribution system. Chlorination was introduced a few years ago and more recently he had undertaken investigations and experiments for the filtration of the city's water on a large scale. One of his most important activities was the long and uphill struggle for the metering of the city's water, to check the enormous and unreasonable waste and the consequent excessive expense for pumping great quantities of water that served no useful purpose. Extensive metering was in progress during 1926, as one of the conditions of the War Department permit for the diversion of water from Lake Michigan to the Chicago Drainage Canal, but Mayor Thompson, now re-elected, openly proposes to repeal the ordinance covering metering.

Among other activities, Mr. Ericson designed a type of bascule bridge that has been used extensively. From 1911 to 1914 he was chairman of the Harbor and Subway Commission which built the $5,000,000 municipal pier and prepared detail plans for a subway system. In 1908 he was offered but declined the post of director of public works at Stockholm. In 1913 the King of Sweden made him a knight of the Royal Order of Vasa. He was a member of the American Society of Civil Engineers, the American Society of Mechanical Engineers and the Western Society of Engineers.

Louisiana Granted Permit to Bridge Chef Menteur Pass

A permit to bridge Chef Menteur Pass, fifteen miles east of New Orleans has been granted to the Louisiana Highway Commission. As this is written an application is on the point of being granted covering a bridge over the Rigolets, a nearby channel between Lake Pontchartrain and Mississippi Sound. These bridges are on the federal-aid road which connects New Orleans and the Mississippi Coast. Failure to proceed with the federal-aid project and the granting of a franchise to a privately-owned toll bridge across Lake Pontchartrain resulted last year in notice being served by the Bureau of Public Roads that all federal money would be withdrawn from Louisiana unless a definite program for completing the federal-aid project was submitted. This led to the ousting of the state highway commission and the formulation of plans by the new commission for toll-free bridges over the Rigolets and Chef Menteur. Each of the bridges is to have 150 ft. of horizontal clearance.

Am. Soc. C. E. Holds Spring Meeting at Asheville

(*Continued from p. 702*)

sessions but at this particular session they were present in a body. Each chapter was represented by a spokesman who told how his chapter was organized and what it did. At the close of their remarks there was a general discussion in which the students and the members participated. It was the universal impression that all of those present, both students and members of the society, were greatly benefited by the atmosphere of co-operation built up by the meeting. Students had a chance to feel that they were a part of the society and to understand, by the close contact of the two days, that their seniors in the society were interested in their welfare and the society members gained a most hopeful opinion of the seriousness with which the youngsters were taking their future.

In the first general session Robert Spurr Weston, of Boston, presented a paper, "Pollution of Streams by Industrial Wastes." It was a general discussion of the problem with particular reference to the conditions of pollution in two New England streams and a description of one plant built to treat the wastes of a textile mill. George W. Fuller, New York City, followed with a paper on "Impounded Waters as a Source of Water Supply," an extended review of the theory and practice of detention reservoirs for domestic supply. He particularly emphasized the fact that while the principles of reservoir design and utilization had been reduced to uniformity over a number of years of practice the exceptions are so great that especial study had to be given each case. The final paper at this session was by H. W. Streeter, U. S. Public Health Service, Cincinnati, on "Polluted Water as a Source of Water Supply." It consisted of an analysis of the efficiency of the different types of treatment plants and then discussed the old question of the relative demand for water treatment and treatment of the sewage which ultimately finds its way to the source of the supply. In some highly polluted waters he finds that the cost of water treatment imposed by such pollution may equal $1 per capita per year as against a possible 90c. per capita per year cost of treatment of the sewage.

Sanitary Division Meeting

In the Sanitary Division meeting E. G. McConnell, Charlotte, N. C., described the sanitary construction for the new Lake Lure development near Asheville. The interesting feature of this work was the decision to take care of the sanitary sewage of the whole development, ultimately to have a population of 35,000, through a gravity-flow cast-iron pipe sewer laid in the bottom of the gorges which were later filled by the waters of the lake which constitutes the attraction of the place. This was selected instead of alternates of separate pumping stations, although it had to be built complete at the start at a cost of some $240,000, whereas the other scheme while costing twice that ultimately could have been built in stages. The sewer line, of Class B cast-iron pipe 8 to 20 in. in diameter, is under a maximum head of 104 ft. It is provided with manholes with screens and removable buckets for screenings removal.

A paper by Clarence E. Keefer, engineer of sewage disposal, Baltimore, described experiments carried out their in pulverizing sewage screenings. This has been used in Berlin but never before in the country. The experiments were made with a motor-driven pulverizer of the type used in industry and upon the regular screenings from Baltimore sewage, the ground pulp being turned into the digestion tanks. It was found that the pulverizer will successfully reduce those screenings with no noticeable effect on the tanks in the disposal of the pulp and with no effect on the pumps. The cost seemed to be equal to or less than collection and incineration. Mr. Keefer said there were many advantages to be seen in the method, and that it warranted study for specific applications.

Power Division Meeting

In the Power Division the general subject was the redesign of existing hydro-electric plants of some years standing. It was opened by a paper by W. S. Lee, Southern Power Co., Charlotte, N. C., who described the redesign of the Catawba station of his company. This station, built 23 years ago, had horizontal shaft triple turbines, 80 per cent efficiency, with rope-driven generators which brought the overall efficiency down to 70.7 per cent for a total rated capacity of 6,600 kw. Increased upstream storage and increased demand brought about the alternative necessity of overhauling the old plant or building a new one. The latter was done. The new plant at much greater head, is rated at 60,000 kw. and has an overall efficiency of around 88 per cent. Mr. Lee pointed out that this instance was illustrative of the great improvement that has taken place in hydro-electric power production in the short space of 20 years and that it presents the great problem of what is obsolescence. He said that for some time it might be necessary to continue to operate less efficient power plants for peaks using the best plants for the base load, getting the best efficiency as and when needed. The discussion consisted mostly in a recitation of similar experience by other engineers.

The Construction Divisions session was devoted to a symposium on textile mills in the South, with special reference to detail development in buildings, plant and housing.

On the last day a full session was devoted to highways. A. C. Rose, Bureau of Public Roads, described the study of subgrades that had been carried out in the Pacific Northwest. Road failure there had been traced to subgrade failure and soil study methods had been applied to the difficulty. Fairly simple tests had been devised whereby the subsoil was graded for moisture content, stability and shrinkage and from the results obtained the reliability of the subgrade could be ascertained and the design of the road slab governed accordingly. The studies so far apply only to this one section of the country, but in the opinion of the author can be extended to other sections with confidence.

An expected debate as to the relative value of asphalt and concrete roads turned out to be merely a statement of the advantages of each type, with no controversial aftermath. Prevost Hubbard, of the Asphalt Association, discussed the asphalt type and Clifford Older, Chicago, concrete. Mr. Hubbard argued against the rich concrete sub-base for asphalt roads, saying it had a tendency to extend the intervals between cracks and to cause larger openings at such cracks with consequent greater damage to the surfacing material. Beam strength in asphalt road sub-base, he said, was not so important for the road structure being flexible was expected to accommodate itself to the sub-surface. He argued, too, for an asphalt base as being the ideal for a flexible road. The advantages of asphalt he listed as easy riding qualities, noiselessness, ease of repair, adaptability to climate, watertightness, temperature resistance, salvage value and adaptability to local material. Some discussion arose as to corrugated sub-bases, but they were generally condemned, and as to waving in asphalt roads, which, it was said by Mr. Hubbard, could be prevented by proper design.

Mr. Older devoted most of his paper to an exposition of a new study of the theory of stress in concrete roads by Prof. Westergaard, of the University of Illinois, who has developed formulas for stress which Mr. Older considers puts the design of the road structure on fully as rigid a basis as for any other structure. He advocated more consideration of modulus of rupture, in road concrete testing, expecially commending crossbreaking tests. He cited as advantages of the concrete road its low tractive resistance, its economy in saving the wear and automobile damage and wear, low maintenance, and ease of design and construction.

U. S. Chamber of Commerce to Meet May 2-5

The 15th annual meeting of the Chamber of Commerce of the United States will be held in Washington from May 2 to 5. The general theme of the annual meeting will be the new business era and five general sessions including the joint general meeting with the Pan-American Commercial Conference will have some of the leading men in public and business life on the program. These include in addition to President Coolidge, Secretary Hoover, President O'Leary of the U. S. Chamber, John Lawrence, Haley Fiske, Silas Strawn, Harry Chandler, George H. Baldwin, and others.

In addition to the general sessions there will be special meetings on agriculture, civic development, finance, insurance, manufacture, natural resources, domestic distribution, and transportation and communication as well as a foreign commerce meeting.

On the evening of May 3 a general joint session with the Pan-American Commercial Conference will be held in the Washington Auditorium and the address will be delivered by President Coolidge.

Washington Notes

SECRETARY WORK of the Interior Department has submitted to his special advisers on the development of the Colorado River, the major problems on which he wishes them to work. They are:

"Whether the federal government by control of water rights has the power to allocate the unappropriated waters of the Colorado River to the basin states and make unnecessary a compact between the states?

"Where the most feasible site is for a dam, now proposed in Black Canyon and known as the Boulder Dam site?

"What the international situation is in relation to the canal now supplying Imperial Valley with water through Mexico and what rights Mexico might have when a storage dam is built?

"Just what benefits the lower basin states will receive from the control of silt and storage of water by the proposed dam?

"Whether or not it is vitally necessary that all revenues from power be confined to repayment to the government of the cost of the dam?"

These major questions make it clear that the advisers will deal with the Colorado River question as a whole and will not confine their work to the Boulder Canyon dam, as some would like. The letter of notification of appointment which went to each adviser, however, contains the statement that it is the desire of Secretary Work "to submit a report to Congress at the forthcoming session, based on your findings, covering the essential features of the Swing-Johnson bill now before that body and the probabilities of reimbursements to the government for its expenditures in this connection."

There is a law prohibiting the use of appropriations for the payment of compensation or expenses of any commission not authorized by Congress. For this reason the attention of each adviser is specifically called to the fact that he is to act as an individual. Each will receive $20.83⅓ per day when actually employed, with a per diem of $6 in lieu of subsistence, for a period not to exceed ninety actual working days. Payments are to be made from the Reclamation Bureau's appropriation for investigations of secondary projects. Actual expenses of transportation also are to be paid. This applies to the appointment of James G. Scrugham, Frank C. Emerson, James R. Garfield, and William F. Durand. It does not apply to Charles W. Waterman, who is a member of the United States Senate.

An effort is being made to induce Secretary Work to appoint other advisers. It is felt that those already named, with the exception of Mr. Garfield, are parties to the Colorado River controversy. At the same time it is recognized that if these advisers should differ with the plan proposed by Senator Johnson and Representative Swing, that it would have much more weight than were it to come from persons known to be in opposition to the Boulder Dam project.

Personal Notes

HENRY GREENE, until recently a junior engineer in the Illinois State Highway Department, is now employed as engineer-designer for the Duke Endowment on work at Duke University, Durham, N. C.

EMMETT H. HENSING, formerly on field work for the Union Pacific R.R., is now located at Alton, Ill., as resident engineer for W. E. Buell & Co., engineer and contractor, Portland, Ore.

GUY BONNEY has entered employment with the engineering department of the Mississippi River Power Co., after previous service in civil engineering work for the Bethlehem Steel Co.

M. U. SNODDERLY is at Fulton, Ky., as resident engineer for Klyce & Harrub, civil engineers, Nashville, Tenn. Mr. Snodderly was assistant engineer of the city of Clarksdale, Miss.

L. M. FAHEY, formerly resident engineer in the work of the Illinois State Highway Department, has located in Evansville, Ind., as assistant in the engineering department of the Chicago & Eastern Illinois Ry.

ORO MCDERMITH, formerly of Denver, Colo., who has been in consulting practice in Washington, is now in San Francisco, Calif., as president of the Derbon Construction Co.; he will continue consulting practice and will also be consultant for the U. S. Bureau of Reclamation.

THE HARMAN ENGINEERING CO., Peoria, Ill., announces that since the death of C. G. Elliott the firm name has been changed from the Elliott & Harman Engineering Co. to the Harman Engineering Co. This business, established originally by Jacob A. Harman, is being continued with three department: (1) General engineering, under the direction of Jacob A. Harman; (2) building construction, under Harris J. Harman and Howard E. Harman; (3) public accounting, under Edward N. Miller. The office at Memphis, Tenn., is continued under the firm name of the Elliott & Harman Engineering Co., for consulting work, under the general direction of Jacob A. Harman and with George J. Schmidt as engineer in charge. The office of the Elliott & Harman Engineering Co. at Washington, D. C., was closed after the death of Mr. Elliott.

LOUIS C. CREW has joined the organization of the Southern International Paper Co. as design engineer and is located at Camden, Ark. This is an industrial corporation that plans, designs, constructs and operates its own plants.

WILLIAM M. ACHESON, who has been appointed chief of the Engineering Division of the Department of Public Works of New York State, as recorded in *Engineering News-Record* April 14, p. 628, was not city engineer of Syracuse, as there stated, but was division engineer, at Syracuse, of the Highway Commission of the Department of Public Works, with which department he has been connected for many years.

Engineering Societies

Calendar

Annual Meetings

NATIONAL CONFERENCE ON CITY PLANNING, New York City; Annual Meeting, Washington, D. C., May 9-11, 1927.

NATIONAL FIRE PROTECTION ASSOCIATION, Boston, Mass.; Annual Meeting, Chicago, Ill., May 9-12, 1927.

SOCIETY OF INDUSTRIAL ENGINEERS, Chicago, Ill.; Annual Meeting, Chicago, Ill., May 24-27, 1927.

CONFERENCE OF STATE SANITARY ENGINEERS, Washington, D. C.; Annual Meeting, Chicago, Ill., June 4-6, 1927.

AMERICAN ASSOCIATION OF ENGINEERS, Chicago, Ill.; Annual Convention, Tulsa, Okla., June 6-8, 1927.

AMERICAN WATER WORKS ASSOCIATION, New York City; Annual Meeting, Chicago, Ill., June 6 to 11, 1927.

AMERICAN SOCIETY FOR TESTING MATERIALS, Philadelphia, Pa.; Annual Meeting, French Lick, Ind., June 20-24, 1927.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION, New York City; Annual Meeting, Orono, Maine, June 27-30, 1927.

The New York Section, American Water Works Association, will hold a convention May 5 and 6 at Oswego. This will be the first two-day meeting ever held by the section and an attractive program has been arranged. Included in it will be addresses and papers by B. C. Little, secretary, A.W.W.A.; George C. Andrews, of Buffalo; C. A. Holmquist, of the New York State Department of Health; William A. McCaffrey, superintendent, Department of Water, Oswego; J. Walter Ackerman, city manager of Watertown; Franklin C. Hopkins, Consolidated Water Co., of Utica; and A. J. Adams, superintendent of water-works, Auburn.

Obituary

HENRY F. MERYWEATHER, former city engineer of Denver, Colo., also an engineer for the board of public works for twenty years, died April 16, aged 77 years. Mr. Meryweather was born in Philadelphia, Pa. He spent ten years in the service of the city of Baltimore, Md., on municipal surveys. In 1880 he went to Denver and for ten years was in charge of preparation of plans, cost estimates, contracts, and construction on many municipal improvements. In 1893 he became chief assistant engineer of the Denver board of public works. In 1913 he was made city engineer of Denver and held that position for a short period.

FRANK A. KUHNLE, assistant city engineer of Toledo, Ohio, died suddenly of heart attack April 15 while at work on a paving job in the city. Mr. Kuhnle had been with the city engineer's department since 1899.

Construction Equipment and Materials

A Section Devoted to What the Manufacturer is Doing for the Engineer and Contractor

Next Road Show in Cleveland

The 1928 Convention and Road Show of the American Road Builders' Association will be held in Cleveland, Ohio, January 9-13. The equipment exposition will be held in Cleveland's auditorium and annex. There is available 45 per cent more area than was used in Chicago, with facilities such as railroad tracks immediately adjacent to the exhibition building. With less expense to the exhibitor, a more satisfactory exhibit may be made and more space secured than in previous years.

An excellent arrangement has been made with the convention board of the Cleveland Chamber of Commerce, by which all hotel reservations will be made through a central bureau, thus assuring everyone satisfactory hotel accommodations. Definite and detailed information as to how reservations should be made will be made public later.

The American Road Builders' Association at the present time is organizing a division of county officials and one of the features of the 1928 convention will be County Officials Day, when papers and discussions covering the problems of county officials will be presented. The entire day will be given up to the activities of county officials and the officers of the day will be selected from county officials.

The great success of Pan American Day, which was participated in by twelve Pan American countries at the 1927 convention, has determined that this day be observed at the 1928 convention, and extra effort will be made to have a larger number of countries represented.

Business Notes

C. W. FERGUSON has been appointed sales manager of the O. K. Clutch & Machinery Co., Columbia, Pa. Mr. Ferguson was formerly advertising manager of the Speeder Machinery Corp., Cedar Rapids, Iowa.

THE ADVERTISEMENT of the American Hume Concrete Pipe Co., Detroit, Mich., appearing in *Engineering News-Record* March 10, page 108, was inserted with the wrong illustration. The one published showed the sewer pipe being laid in Michigan, while the correct one should have shown a pressure line in Southern California as noted in the published caption.

WILLIAM CRAMP & SONS SHIP AND ENGINE BUILDING CO., Philadelphia, has announced that it will discontinue shipbuilding because of the general curtailment of naval construction and the continued depression in merchants shipbuilding. The manufacturing operations of the company's subsidiaries, the De LaVergne Machine Co., I. P. Morris Corp., Pelton Water Wheel Co., Federal Steel Foundry Co., Cramp Brass & Iron Foundry Co., and Cramp Engine Mfg. Co. will be continued. It is said that the business of the subsidiary companies is in a sound and prosperous condition and is capable of considerable expansion.

YEOMANS BROTHERS CO., Chicago, announces the appointment of two new representatives: Philadelphia, A. R. Amos, Jr., 1014 Harrison Bldg., and Des Moines, Iowa, C. L. Knehr, 521 Hubbell Building.

L. P. FEUSTMAN, vice-president of the Worthington Pump and Machinery Co., died April 7 at his home in New York City.

New Developments

New 1-Yd. Shovel Built Unusually Strong

Contrasted to the policy of changing a ¾-yd. or a ⅞-yd. shovel to meet 1-yd. requirements, the Thew Shovel Co., Lorain, Ohio, has converted its 1¼-yd. machine, known as the Lorain-75,

into a 1-yd. machine, known as the Lorain-60. This new machine is the Lorain-75 in everything except dipper capacity, power plant, power take-off and counterweight. In no other details have any changes been made. The new shovel can also be converted to a crane handling a 50-ft. boom.

New Half-Yard Rope Crowd Shovel

A new shovel was introduced by the Byers Machine Co., Ravenna, Ohio, at the Chicago Road Show and designated as model 27-R. The boom is steel and of box girder construction. The dipper stick is of the double type, working in a saddle block outside of the boom. The boom is made of seasoned oak timber completely armored with steel plates.

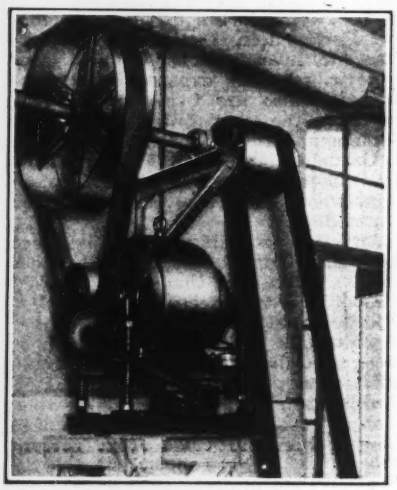
The shovel is of the rope crowd type and has a capacity of ½ yd. heaped measure. The rope crowd is accomplished by means of a drum actuated by a clutch at each end; the clutches operate the drum in opposite directions, and are controlled by one lever. Shovel

operation is standard with three levers, and foot brake although an extra foot brake is provided to assist in holding the dipper stick in any position as well as for use with the back drum when some other attachment is used. Standard attachments consisting of skimmer, ditcher, clamshell and backfiller ordinarily used on the company's Bear Cat machine fit the new shovel without change.

Spring-Mounted Motor Base Improves Motor Efficiency

The performance of electric motors when mounted on a so-called tension motor base for vertical, horizontal or inclined drives is claimed to be smooth, vibrationless, noiseless and exceptionally efficient. Although these motor bases have been used in foreign countries for some time, they have only recently been introduced into the United States. They are manufactured by the Tension Motor Base Co., New York City, and are being sold by the Nilson-Miller Corp., Hoboken, N. J.

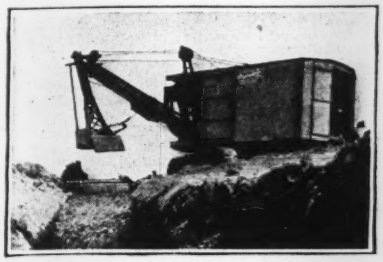
The tension motor base consists of a movable platform on which the motor is mounted, supported at each corner by steel springs whose tension is adjusted to suit the operating conditions. A counterweight is connected to a pin-



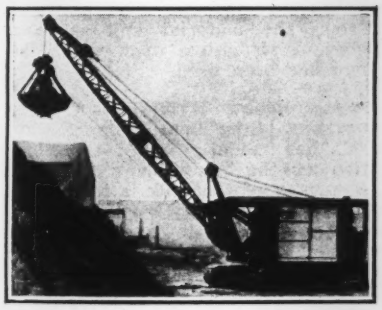
ion shaft engaging a rack which passes downward from the center of the spring-supported platform. The action of the lever is designed to produce a constantly uniform tension on the driving belt. Throwing the lever and counterweight to the opposite side disengages the motor instantly. An advantage is that the motor may be started without load and when up to speed the load gradually applied by adding the counterweight. It is claimed that economy in floor space is made possible since the tension motor base permits shafts being operated on short center distances and with rather high pulley ratios.

Crane and Ditcher Booms for Telescoping Boom Shovel

In order to widen the scope of usefulness for its telescoping boom shovel, the Star Drilling Machine Co., Akron,



Ohio, has developed attachments for converting it to a crane or a pull ditcher shovel. The shovel, which was introduced during the latter part of 1925 and was then described in *Engineering News-Record*, Dec. 31, 1925, p. 1091, is of novel design, the bucket being attached to the end of the boom



which moves in and out from the body of the shovel. The usual shipper shaft arrangement is thus eliminated.

A number of advantages are claimed for the new ditcher attachment. In the first place, because of the telescope rack, the operator can easily regulate his boom reach and dumping clearances. It is further claimed that the bucket is always at the most efficient digging angle which does not permit the heel of the bucket to drag at any time during the stroke. A maximum clearance of 12 to 14 ft. can be attained and a ditch 18 ft. deep can be excavated. A backfiller attachment is also available for this pull-in shovel. It is shown attached in the upper illustration. The maximum reach on the center pin is 31 feet.

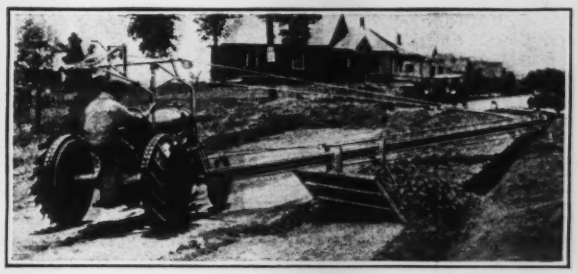
The crane boom accommodates a ½-yd. digging bucket or a ¾-yd. material bucket. Either the regular ditcher boom, 19 ft. long, or a special 30-ft. crane boom as shown in the lower illustration may be used. The shovel is full revolving. With the longer boom a dumping elevation of 20 to 24 ft. is possible.

New Adjustable Shore

Distinctive features of the new shore, designed by the Dayton Sure Grip & Shore Co., Dayton, Ohio, are its speed in erection and adjusting, its automatic pin adjustments which will not settle under load, and its rugged construction. Each shore is equipped with individual jack, thus requiring no loose tools for its operation. The standard size shore has a range adjustment of from 8 ft. to 14 ft. and weighs 55 lb.

New Type Backfiller Has Many Advantages

A piece of auxiliary equipment has recently been developed by the Baker Steel & Machinery Co., Omaha, Nebraska, for the standard Baker-Fordson one-man-operated backfiller. It is known as a "Kicker" and consists of an auxiliary boom and bucket which reverses the usual action of the backfiller by pushing the dirt into the ditch from the same side on which the ma-



chine is located, instead of pulling it in from the opposite side.

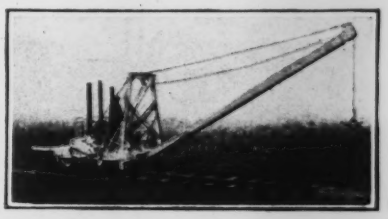
It is claimed that the new arrangement is especially handy for backfilling ditches that are laid near the curb, where the spoil bank is thrown toward the center of the street. The boom may be set at any desired angle.

The "Kicker" boom and bucket are interchangeable with the standard boom and bucket, not more than 15 minutes being required for the change. The "Kicker" boom consists of two channels, 16 feet long, on which runs a roller carriage to which is hinged the 48-in. scraper.

Crane Boom Completes Machine for General Track Work

The machine shown in the illustration was first developed as a track shifter and later was improved by the addition of a leveling device, consisting of two spuds by means of which the track could be lifted vertically to permit surfacing operations. This same machine, which is manufactured by the Nordberg Mfg. Co., Milwaukee, Wis., has now been further improved by the addition of a hoist and boom, making the machine equally suitable for rail laying.

In addition to being a rather universal machine for track work, it can also



be used for transporting men and materials, since it is self-propelling. The machine is 10 ft. long by 7 ft. wide by 8 ft. wide and weighs 8,800 lb.

Building Mixers Improved to Meet Mix Requirements

The present tendency to require less water in the mix and to keep the batch mixing for a longer period, often 1½ to 2 min., places new demands upon concrete mixing equipment. The new 7-S and 10-S building mixers, manufactured by the Knickerbocker Co., Jackson, Mich., have been designed to enable the contractor to conform to new mixing specifications and still not cut down his daily average.

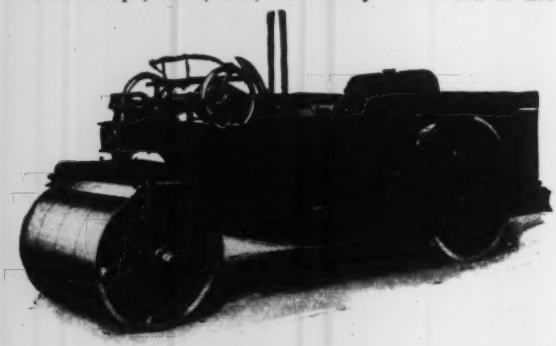
In the new 7-S the diameter of the drum has been increased to 50 in., permitting a 22½-in. opening on the charging side and a 20-in. opening on the discharge side. The higher drum has made possible an increase in the size and number of mixing buckets. The drum head is a $\frac{3}{16}$-in. steel plate and is welded to a $\frac{3}{16}$-in. steel shell. The drum revolves at a speed of 20 r.p.m. in both sizes of building mixers.

The discharge chute is set at an angle of 50 deg. which insures a speedy

discharge. Bevel gears actuated by a handwheel from either side of the mixer have been substituted for levers in order to make the operation of the discharge chute easier. The frame of the 7-S has been made heavier, being built up of 7-in. steel channels arc welded and reinforced with steel gussets. Similar improvements have been made on the 10-S with the parts proportionately larger and heavier.

New 8-Ton Gasoline Roller

Although for many years manufacturers of steam rollers, the Erie Machine Shops, Erie, Pa., have only recently developed a gasoline roller. This new unit is an 8-ton size, tandem design, powered by a Waukesha engine and using Twin Disc heavy-duty clutches, ball and roller bearings, and having all drive gears cut, hardened and ground. The power unit is mounted on a cast-iron base insuring rigidness throughout. The over-all height of the roller is made especially low with high clearance for rolling close to the curb.



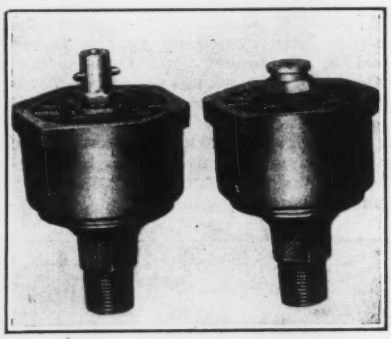
Huge Roller Bearing for Portland Cement Mill

The largest bearings ever built by the Timken Roller Bearing Co., Canton, Ohio, have just been completed and shipped to the Allis-Chalmers Mfg. Co., Milwaukee, who will install them in a portland cement plant. The bearings have a bore of 42 in. and an outside diameter of 62 5/16 in. At 30 r.p.m., the bearings have a capacity of 2,750,000 lb. The weight of each bearing is over two tons.

The cement grinding mill upon which they will be installed has a diameter of 10 ft. and is 45 ft. long. One of the advantages of this bearing will be a material reduction in the overall length of the mill, since the width of the bearing is only 13¾ in.

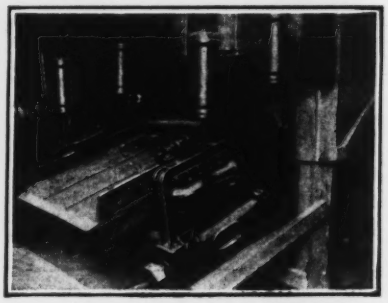
New Compression Grease Cup

A malleable-iron forced-feed or compression grease cup with hexagonal head to give an easy purchase for a wrench or a good grip for the hand has been put on the market by the Link-Belt Co., Chicago, under the name of the Hex-Top grease cup. It is adapted for general use on machinery, and by means of a grease gun the filling is done easily even in positions awkward of access. It is stated that with machinery having numerous such cups it is economical to fill them all at one time with the grease gun, the hexagon top of the cup being turned high up but not entirely unscrewed or removed. In this way the filling can be done without waste or inconvenience. The cup holds a good reserve for use with the compression feature, so that an occasional screwing down of the top is sufficient to maintain ample lubrication until a general refilling of all the cups is required. If additional lubrication is necessary at any point, as in case of a bearing getting warm, a few turns of the hexagonal top of the grease cup will give the desired supply.



Noiseless Vibrating Screen

An electrically operated vibrating screen is a new product of the Traylor Vibrator Co., Denver, Colo. Uniform vibration is secured by the use of stators and rotors energized by alternating current; a small motor generator set is a part of the equipment. The rotors are assembled on the vibrator members as shown in the illustration. Enclosed shock absorbers do away with the vibration between the screen and the superstructure or screen platform supports. There are no bearings or moving parts and the operation of the screen is therefore noiseless. Another advantage is that no oil or grease is necessary. The absence of moving parts, bumper blocks and bearings contributes to the light weight of the screen. It is designed to operate on 110, 220 or 440-volt, 60 cycle a.c. A smaller machine is made in a laboratory size, weighing only 20 pounds.



Manufacturers and Trade Associations

Calendar

Annual Meetings

AMERICAN WELDING SOCIETY, New York; Annual meeting, New York, April 27-29.

NATIONAL LUMBER MFRS. ASSOCIATION, Washington, D. C.; Annual meeting, Chicago, April 28-29.

NATIONAL LIME MANUFACTURERS' ASSOCIATION, Chicago; Annual meeting Chicago, April 28-29.

WATERWORKS MANUFACTURERS ASSOCIATION, New York; Annual meeting, Chicago, June 6-10.

New Publications

Waterproofing — MASONRY WATERPROOFING CO., INC., 307 Herkimer St., Brooklyn, N. Y., has made available a folder explaining its "Larutan system" of membrane waterproofing which consists of a layer of cloth fabric saturated and coated with a special bituminous compound. The pamphlet describes the material and shows a number of structures upon which the system has been used.

Oxyacetylene Welding — OXWELD ACETYLENE CO., Long Island City, N. Y., has recently issued the ninth edition of the "Oxwelder's Manual." The purpose of the book is to instruct purchasers of oxyacetylene welding and cutting apparatus in its use. It is available to purchasers of the equipment free and to others at a charge of $1.

Motor Truck Fleets—INDIANA TRUCK CORP., Marion, Ind., has issued a booklet containing a list of the individual concerns, corporations, state highway departments and municipalities owning fleets of trucks. Several pages of illustrations of these trucks are given.

Silent Gears — GENERAL ELECTRIC CO., Schenectady, N. Y., in catalog GEA-482, discusses its gears which go by the copyright names of Fabroil and Textoil. Both are made from spinnable textile fibers. The booklet goes into the history, manufacture, characteristics, and application of these gears to various types of machinery, the catalog being well illustrated with silent gear installations. A complete table is given of the available sizes of gears.

Business Side of Construction

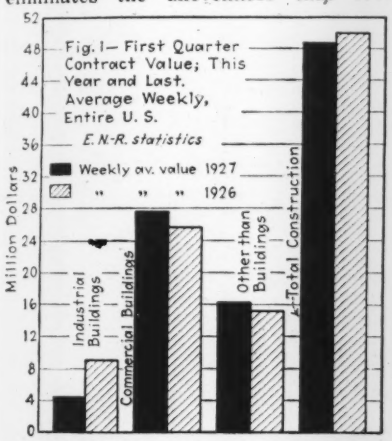
Facts and Events That Affect Cost and Volume

Contract Letting Slower Than in 1926

Deficit Due to Falling-Off in Residential Awards—Heavy Construction Gains

CONSTRUCTION operations, over the entire country, as measured by money value of contracts let, may be found to have lagged behind the first quarter of 1926 but the deficit for the Jan. 1-April 1, 1927 period will not amount to more than 3½ per cent.

This statement is based on a comparison of weekly averages of contracts awarded for the first three months of 1927, with the corresponding period last year. The weekly comparison eliminates the unevenness that frequently develops where monthly totals are involved.



Commercial buildings and construction other than buildings are conspicuously ahead of a year ago, as shown by the diagram. The class designated as commercial includes also educational, religious, institutional and other buildings not industrial or small residential jobs.

Construction other than buildings comprises water-works, sewers, bridges, excavations, drainage, irrigation, levee, river and harbor projects, streets and roads, Federal Government work and various unclassified contracts.

Residential buildings mentioned in Fig. 2 are from the F. W. Dodge Corp., and cover projects of all sizes in this class. All other data are from *Engineering News-Record.*

In order to get a proper understanding of the money values shown in Fig. 1 it is necessary to bear in mind the minimum adopted in each class, as follows: Water-works and excavations, $15,000; other public works, $25,000; industrial buildings, $40,000; commercial and other buildings, $150,000.

The total for the first quarter of 1926 contains the Susquehanna Power Company's project at Conowingo, Md., valued at $52,000,000. This job was awarded to Stone & Webster, Inc., and sub-let to the Arundel Corp. The Conowingo project is included in the industrial weekly average in 1926 (Fig. 1) giving it an abnormal advantage over figures for the first quarter this year. In Fig. 2, however, the big power-plant contract has been omitted from the industrial classification, giving 1926 figures a very small lead, compared with first quarter 1927.

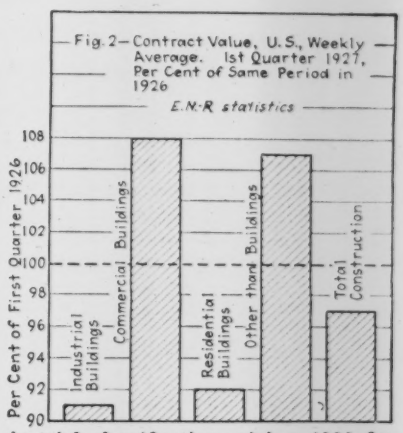


Fig. 2—Contract Value, U. S., Weekly Average. 1st Quarter 1927, Per Cent of Same Period in 1926
E.N.R. statistics
Per Cent of First Quarter 1926
Industrial Buildings
Commercial Buildings
Residential Buildings
Other than Buildings
Total Construction

This Week's Contracts—Week Ago—Same Week Last Year

Minimum costs observed are: $15,000 for water-works and excavations; $25,000 for other public works; $40,000 for industrial and $150,000 for commercial, educational, religious and other buildings.

MONEY VALUE OF CONTRACTS LET—ENTIRE U. S.

Week ended	Public Work (000) omitted	Private Work (000) omitted	Total Contracts (000) omitted
Apr. 28, 1927	$24,218	$34,353	$58,571
Apr. 21, 1927	24,494	50,628	75,122
Apr. 29, 1926	18,722	26,101	44,823
Jan. 1 to date			
1927	308,203	569,681	877,884
1926	301,483	578,662	880,145

Municipal Improvement Bond Sales For First Quarter Below Year Ago

STATE and municipal bond disposals for the first quarter of 1927, according to the *Commercial and Financial Chronicle* are appreciably below the first quarter of 1926. Only $320,293,927 was borrowed the first three months of this year as compared to $332,437,479 borrowed last year. The total for March, 1927 reached $80,102,681 as against $116,823,292 in March, 1926 and $73,682,778 in February, 1927. Following are a few representative issues:

COST OF FINANCING COUNTY AND MUNICIPAL IMPROVEMENTS AS SHOWN BY MARCH BOND SALES

County	Purpose	Amount	Rate Per Cent	Sold for	Basis	Dated	Maturity	Purchased by
Allen, Kan.	Road	$140,000	4½	101.10	....	Feb. 1	1928-47	Stern Bros. & Co. of Kansas City
Broward, Fla.	Court House and Jail	300,000	5	95	....	.....	.....	Brown-Cummer Co. of Wichita and others
Cook, Ill.	Jail	3,400,000	4	99.377	....	June 1	....	A. B. Leach & Co. of Chicago and others
Delaware, O.	Road	72,800	5	101.88	....	Mar. 8	1928-36	First Citizen's Corp. of Columbus
Greenville, S. C.	Highway	600,000	4½	100.059	4.49	Feb. 1	1929-43	Bankers' Trust Co. of New York City and others
Harris, Tex.	Road	1,450,000	5	100.87	4.92	Apr. 10	1928-56	Eldredge & Co. of New York City and others
Kent, Del.	Road	200,000	4½	.....	....	Oct. 1	1943-52	Farmer's Bank of Dover
Lafayette, Fla.	Road and Bridge	250,000	5	90	7.07	.....	1931-37	Spitzer, Rorick & Co. of Toledo
Muscogee, Ga.	Road	323,000	4½	102.765	4.20	Apr. 1	1927-56	Trust Co. of Georgia of Atlanta and others
Oakland, Mich.	Road	2,140,000	..	100.528	....	.....	1929-38	Watling, Lerchen & Hayes of Detroit and others
Suffolk, N. Y.	Highway	378,000	4½	105.59	3.98	Apr. 1	1928-57	Baor Bros. & Co. of New York and others
Township								
Boardman, O.	Fire Protection	15,000	5	100.300	4.87	Apr. 1	1928-31	Davies-Bertram Co. of Cincinnati
Delaware, N. J.	Sewer, Road and Street	105,700	5½	.....	....	.....	6-10 yrs.	M. M. Freeman & Co. of Philadelphia
Lake, Ind.	Road	34,000	4½	101.77	....	Oct. 1	.....	First National Bank of Fort Wayne
Municipal								
Akron, N. Y.	Water	252,000	4½	102.28	4.32	Mar. 1	1931-66	Batchelder, Wack & Co. of New York and others
Birmingham, Ala.	Public Improvement	400,000	4½	100.92	4.30	Apr. 1	1928-37	Eldredge & Co. of New York
Bronxville, N. Y.	Street Improvement	75,000	4½	102.09	4.18	Dec. 1	1927-41	Phelps, Fenn & Co. of New York
Cleveland, O.	Paving and Sewer / Fire Alarm	1,000,000 / 275,000	..	101.44	3.95	Apr. 1	1928.42	Roosevelt & Son of New York
Dubuque, Ia.	Dock	65,000	4½	101.12	4.30	Jan. 1	1929-37	George M. Bechtel & Co. of Davenport
East Providence, R. I.	School	500,000	4½	100.163	4.24	Apr. 1	1928-57	National City Co. of New York and others
Fort Wayne, Ind.	School Improvement	100,000	4½	101.47	4.07	Mar. 1	1928-47	Lincoln National Bank of Fort Wayne
Greenwood, Miss.	Flood Protection	15,000	5½	101.50	5.02	Feb. 1	1928-42	Flora and Esther Reiman of Greenwood
Herkimer, N. Y.	Paving	16,390	5	100	.....	.....	1927-31	First National Bank of Herkimer
Kings Mountain, N. C.	Sewer, Water and Street	40,000	5½	101.87	5.07	Jan. 1	1929-59	Caldwell & Co. of Washville
New London, Conn.	Public Improvement	50,000	4½	101.30	3.99	Apr. 1	1928-37	Winthrop Trust Co. of New London
South Amboy, N. J.	Water	49,000	5	102.57	4.65	Apr. 1	1928-44	M. M. Freeman & Co. of Philadelphia

Weekly Construction Market

A DECIDEDLY firmer market with upward price tendencies is evident in brick, lime, plaster and paint materials at the principal centers of distribution, throughout the country. This movement is seasonal to a certain extent and is due also to restriction of production, low reserve stocks, and a current demand that can accommodate all available material.

The opposite condition exists with regard to certain other materials going, perhaps, into the same structures as those showing an excess of advances over declines. Those in which an obviously downward trend exists are concrete aggregates, window glass, shingles, Douglas fir flooring, No. 1 common boards, sheathing, and pine two-by-fours. Prices show more of a tendency to decline than to rise in steel structurals, reinforcing bars, small sized steel pipe, wire nails, and cement, in other than the large industrial areas. Demand, in the materials showing declines, is in large volume but not sufficiently so to overlap the present high rate of mill output.

The three-day closing down of some of the larger lumber mills in the Pacific Northwest, in an effort to stabilize the market, has not yet resulted in higher price quotations. Reports from mid-western lumber yards, wholesale and retail, indicate reserve stocks standing at slightly above normal for the season, owing to a smaller volume of shipments than were expected earlier in the year.

	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Steel Products									
Structural shapes, 100 lb.	$3.34	$3.80	$4.15	$3.10	$3.35	$3.87½	$3.00	$3.15	$3.80
Structural rivets, 100 lb.	5.00	3.80	4.75	3.50	3.75	4.65	5.00	3.75	....
Reinforcing bars, ½ in. up, 100 lb.	3.24	2.80	2.75	2.30@2.50	2.87½	3.87½	2.95	2.75	3.57
Steel pipe, black, 2½ to 6 in. lap, discount	48%	54%	54%	51%	48%	41%	42@53.8%	48%	37.83
Cast-iron pipe, 6 in. and over, ton	48.60	42.00	—54.00	46.20	48.50	56.50	+56.00	55.00	62.50
Concreting Material									
Cement without bags, bbl.	2.35@2.50	2.35	2.05	2.05	2.22	2.85	2.51	2.65	1.15
Gravel, ¾ in., cu.yd.	1.75	1.90	2.38	1.85	1.65	1.90	1.80	1.25	1.90
Sand, cu.yd.	1.00	1.60	2.00	1.65	1.25	1.00	1.40	1.25	1.35
Crushed stone, ¾ in., cu.yd.	1.94	2.50	2.83	1.87½	1.75	2.50	1.70	3.00	2.00
Miscellaneous									
Pine, 3x12 to 12x12, 20 ft. and under, M.ft.	—59@61	34.00	56.00	40.50	38.75	33.25	27.00	23.00	50.00
Lime, finishing, hydrated, ton.	18 20	22.00	19 00	20.00	25.50	24.00	27.50	24.00	21.00
Lime, common, lump, per bbl.	2.10@3.00	1.50	1.82	1.50	1.70	2.70	1.60	2.80	10.00
Common brick, delivered, 1,000.	19@20	12.00	14.10	12.00	13.75	9@10	14.00	14.00	20.25
Hollow building tile, 4x12x12, per block	Not used	.0895	.112	.076	.072	.085	.....	.10	.10
Hollow partition tile 4x12x12, per block	.1027	.0895	.112	.076	.072	.085	.108	.09	.08
Linseed oil, raw, 5 bbl. lots, per 7½-lb. gal.	+.85½	.89¼	1.10	+.85	+.95¼	1.06	.89	.95	1.05
Common Labor									
Common labor, union, hour.	.90⅜	.30	....	.90	.....	.50@.55	.60	.62½	.....
Common labor, non-union, hour.	.....	.25	.30@.50	....	.45@.60	.40@.45	.50	.62½	.25@.30

Explanation of Prices—Prices are to contractors in carload lots unless other quantities are specified. Increases or decreases from previous quotations are indicated by + or — signs. For steel pipe, the prevailing discount from list price is given; 45-5% means a discount of 45 and 5 per cent. L.c.l. is less than carload lots.

New York quotations delivered, except sand, gravel and crushed stone, alongside dock; common lump lime in 280-lb. bbl. net and hydrated lime, f.o.b. cars; tile "on trucks"; linseed oil and cast-iron pipe f.o.b. Reinforcing bars (billet steel) and shapes delivered to job in less-than-carload lots.

Labor — Cement and concrete laborers' rate, $1.06¼; building laborers, 90⅜c.

Chicago quotes hydrated lime in 50-lb. bags; common lump lime per 180-lb. net. Lumber, sand, gravel and stone f.o.b.; price on fir is quoted instead of pine. Reinforcing bars (billet steel) f.o.b. warehouse in carload lots; shapes, less-than-carload lots.

Minneapolis quotes on fir instead of pine. Brick, sand and hollow tile delivered. Cement on cars. Gravel and crushed stone quoted at pit. Bars (billet steel) at warehouse in carload lots; shapes, less-than-carload lots. Steel pipe under 3-in., 51% off list.

THIS limited price list is published weekly for the purpose of giving current prices on the principal construction materials, and of noting important price changes on the less important materials. Moreover, only the chief cities are quoted.

Valuable suggestions on costs of work can be had by noting actual biddings as reported in our Construction News section.

The first issue of each month carries complete quotations for all construction materials and for the important cities. The last complete list will be found in the issue of Apr. 7, the next on May 5.

Denver quotes on fir instead of pine. Cement "on tracks"; gravel and sand at pit; stone on cars; lime, brick, hollow tile and lumber on job. Tile price is at warehouse. Linseed oil, delivered in wooden bbl. Common lump lime per 180-lb. net. Bars (billet steel) and shapes, l.c.l.

Atlanta quotes sand, stone and gravel per ton instead of cu.yd. Common lump lime per 180-lb. net. Bars (billet steel) f.o.b. in carload lots; shapes, l.c.l.

Dallas quotes lime per 180-lb. bbl. Cement, cast-iron pipe and crushed stone f.o.b. cars, other materials delivered. Bars (billet steel) carlots; shapes, l.c.l.

San Francisco quotes on Heath tile, size 5½ x 8 x 11½. Prices are all f.o.b. warehouses except C.-I. pipe, which is mill price plus freight to railway depot at any terminal. Common lump lime per 180-lb. net. Lumber prices are to dealers in yards at San Francisco, for No. 1 fir, common. Bars (billet steel) f.o.b. in carload lots; shapes, l.c.l.

Seattle quotes on Douglas fir (delivered) instead of pine. Lump finishing lime per 180-lb. net. Brick and hollow building tile delivered. Hydrated lime in paper sacks. Sand and gravel at bunkers. Bars (billet steel) carlots; shapes, l.c.l.

Montreal quotes on fir lumber. Sand, stone, gravel and lump lime per ton. Stone and tile are delivered; sand, gravel, lime and cement on siding; steel and pipe at warehouse. Hollow tile per ft. Cement price is in Canadian funds (the Canadian dollar stands at 100.15¢). Bag charge is 80c. per bbl. Discount of 10c. per bbl. for payment within 20 days from date of shipment. Steel pipe per 100 ft. net; 2½ in., $37.83. Bars (billet steel) and shapes, in carload lots.

Unit Prices

For actual prices bid on materials in place at various construction jobs throughout the country, see p. 88-90 of this issue, following Construction News.

On Apr. 1, 1927

E. N.-R. Construction Cost Index Number 209.00

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For Explanation and Details of Indexes Since 1913 See the First Issue of Every Month